BLACKSTONE RIVER FLOOD CONTROL PROJECT

WORCESTER DIVERSION

BLACKSTONE RIVER MASSACHUSETTS

DEFINITE PROJECT REPORT

WAR DEPARTMENT, CORPS OF ENGINEERS, U. S. ARMY
U. S. ENGINEER OFFICE, PROVIDENCE, RHODE ISLAND

OCTOBER 1946

DEFINITE PROJECT REPORT

ON

WORCESTER DIVERSION

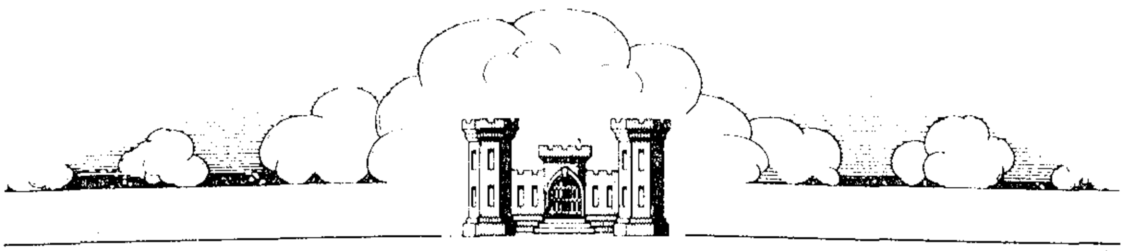
BLACKSTONE RIVER BASIN
MASSACHUSETTS

PREPARED IN THE U.S. ENGINEER OFFICE

PROVIDENCE, R.I. DATED ____________

APPROVED BY THE CHIEF OF ENGINEERS ________ 194__

E.D. FILE ____________



WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY
U. S. ENGINEER OFFICE PROVIDENCE, R.I.

REVISIONS		
DATE	NEW PAGES OR DRAWINGS	DATE APPROVED BY C. OF E.

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

BLACKSTONE RIVER, MASSACHUSETTS

Sept. 1946

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

SECTION 1

TABLE OF CONTENTS

BASIS FOR DESIGN

APPENDICES

I. Hydrology

II. Geology

III. Soil Data and Analysis

IV. Hydraulic Design

V. Structural Design (Structures and Improvements)

VI. Hydroelectric Power

VII-A. Relocations

VII-B. Real Estate Planning

VIII. Availability of Construction Materials

IX. Special Studies

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

LIST OF PLATES

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

SECTION 2

PERTINENT DATA

LOCATION OF DIVERSION

Diversion extends from Leesville Pond on Kettle Brook at Mile 53 above the mouth of the Blackstone River to the Blackstone River at the mouth of Hull Brook (Mile 46). Project is located in the Towns of Auburn and Millbury, Worcester County, Massachusetts, approximately 28 miles airline distance west of Boston, Massachusetts, 28 miles northwest of Providence, Rhode Island, and 4.5 miles south of Worcester, Massachusetts.

DRAINAGE AREAS

Leesville Pond (Diversion intake)	31.3
Kettle Brook at Gaging Station	31.3
Middle River at Junction Tatnuck Brook and Kettle Brook	51
Blackstone River at Hull Brook	69.8
Blackstone River at mouth	540

STREAM FLOW

Record of U.S.G.S. Gaging Station on Kettle Brook (31.3 Sq.Mi.)

	Acre Feet	Cu. Ft. Per Sec.	Cu.Ft. Per Sec. per Sq. Mi.
Average Annual (5 years 1939-1943)	26,260	35.9	1.15
Maximum Year (1940)	33,590	45.9	1.47
Minimum Year (1941)	15,270	20.9	0.67
Maximum Month (April 1940)	11,820	197	6.30
Minimum Month (Oct. 1939)	560	9.01	0.29

Maximum Flood of Record - March 1936

Location	Drainage Area sq. miles	Peak Discharge cu.ft. per sec.
Kettle Brook at Worcester	31.3	2520
Blackstone at Northbridge	137	7510
Blackstone at Blackstone	354	13600
Blackstone at Woonsocket	416	15000

DESIGN FLOOD DATA

Total volume of rainfall, inches	15.5
Total volume of runoff, inches	10.3
Duration of flood, days	5
Maximum inflow at Leesville Pond, c.f.s.	5,600
Maximum flow in Diversion, c.f.s.	5,600
Flow over Leesville Dam	0
Maximum flow in Blackstone River at mouth Hull Brook, c.f.s.	11,400

TUNNEL

Type - Circular concrete in earth and concrete lined circular in rock.

Length, feet	4,520
Diameter in feet	16
Elevation invert at intake portal	455.0
Elevation invert at outlet portal	407.0
Design Capacity, c.f.s.	5,600

Stilling Basin

Type	Concrete lined with end weir.
Length	220 ft.
Bottom elev.	405.0 m.s.l.
Bottom width	16 ft. to 40 ft.

INTAKE

Type	Gate controlled
Sill elevation	455
Headwater elevation under design flood	484.5

Service Gates

Number	3
Size	6' x 12'
Type	Electrically operated cast iron sluice gates.
Combined Capacity	5,880 c.f.s.

Emergency Gate

Number	1
Size	6' x 12'
Type	Crane operated truck steel gate.

CHANNEL

Type	Open in rock and earth.
Length	10,300'
Bottom width	40' in rock - 50' in earth
Slopes	4 on 1 in rock - 1 on 2-1/2 in earth

ESTIMATED PROJECT COST (Current prices)

Federal Cost

Total construction cost	$3,353,000

City of Worcester

Relocations	245,000
Land Acquisition	76,000
Total	321,000

Total Project Cost	$3,674,000

SECTION 3

PROJECT AUTHORIZATION

1. The flood protection plan for the Blackstone River Basin was approved by the Flood Control Act, approved 22 December 1944, (Public Law 534 - 78th Congress), which reads, in part, as follows:

"Sec. 10. That the following works of improvement for the benefit of navigation and the control of destructive flood waters and other purposes are hereby adopted and authorized***

"*** The project on the Blackstone River for local flood protection at Worcester, Massachusetts, is hereby authorized substantially in accordance with the Recommendation of the Chief of Engineers in House Document No. 624, 78th Congress, 2d Session, at an estimated cost of $2,232,000."

2. Local cooperation is required as recommended by the Chief of Engineers in House Document No. 624, 78th Congress, 2d Session, which reads, in part, as follows:

"Par. 14. ***, provided local interests give assurances satisfactory to the Secretary of War that they will furnish without cost to the United States all lands, easements, and rights-of-way; hold and save the United States free from claims for damages due to the construction works; maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of War; construct two highway bridges and one railroad bridge across the Worcester Diversion channel; ***."

SECTION 4

INVESTIGATIONS

3. Preliminary Report Authorization. - The Flood Control Act approved 22 June 1936 (Public No. 738, 74th Congress) authorized a preliminary examination and survey for flood control of the Blackstone River and its tributaries, Rhode Island and Massachusetts. The preliminary examination report was submitted on 31 October 1936.

4. Report on Survey. - A survey report was directed to be made by Congress upon the recommendation of the Board of Engineers for Rivers and Harbors made on 21 December 1936. The Chief of Engineers in a letter dated 30 July 1937 authorized the survey report. A report entitled, "Report on Survey for Flood Control, Blackstone River," dated 30 March 1940, by the United States Engineer Office, Providence, Rhode Island, was submitted to the Chief of Engineers, U. S. Army, Washington, D. C.

b. The report of the Board of Engineers for Rivers and Harbors on survey reports on the Blackstone River is contained in House Document No. 624, 78th Congress.

5. Definite Project Report Investigations. - The definite project proposal is based on detailed surveys of the tunnel and channel areas; adequate borings and seismic investigations of tunnel, channel and structure foundation materials; detailed hydraulic and hydrological studies; and basic design studies as outlined herein.

6. Public Hearings. - A public hearing was held at Woonsocket, Rhode Island, on 3 August 1939, in connection with the survey report. No public hearing was held for this specific project.

SECTION 5

LOCAL COOPERATION

7. Local cooperation required by the authorizing legislation is as follows:

That local interests will furnish without cost to the United States all lands, easements, and rights-of-way; hold and save the United States free from claims for damages due to the construction works; maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of War; construct two highway bridges and one railroad bridge across the Worcester Diversion Channel; relocate three telephone service poles; lower a petroleum pipe line and care for an 8-inch water main under the present bridge on Greenwood Street. The estimated total cost to the City of Worcester is $321,000.

8. Informal assurances have been requested on the items stated in paragraph 7. A brochure has been prepared and sent to the officials of the City of Worcester to assist the city in considering this project. No reply has, as yet, been received. On the basis of informal discussions with City officials, it is anticipated that the assurances of local cooperation will be forthcoming. However, there is no legal basis on which the city can acquire lands outside the city limits and an enabling act by the State Legislature will be required. It is understood that an enabling act will be presented at the next regular session of the Massachusetts Legislature which is presently scheduled for 2 January 1947.

9. The names, titles and addresses of the principal municipal officers responsible for the fulfillment of local cooperation requirements are as follows:

Honorable Charles F. Jeff Sullivan
Mayor, City of Worcester
City Hall
Worcester, Massachusetts

Mr. William B. Lynch, Chairman
Finance Committee
10 May Street
Worcester, Massachusetts

SECTION 6

LOCATION OF PROJECT AND TRIBUTARY AREA

10. Location (See Plate No. 2). - Kettle and Tatnuck Brooks draining areas of 34 and 17 square miles, respectively, rise in the northernmost part of the Blackstone River basin and drain generally southward joining in upper Worcester to form the Middle River. The Middle River flows through the city and joins Mill Brook to form the Blackstone River in the southerly part of the city. Hull Brook joins the Blackstone River approximately 1/4 mile downstream of the city limits.

11. The intake of the Worcester Diversion is located 2.25 miles upstream from the junction of Kettle and Tatnuck Brooks on the easterly edge of Leesville Pond, an artificial pond on Kettle Brook. From the intake the diversion passes southerly in a tunnel through Pakachoag Hill thence in an open channel along the bed of Hull Brook to the Blackstone River. The intake is located 4.5 miles south of the center of Worcester and the entire project lies in the towns of Auburn and Millbury, Worcester County, Massachusetts.

12. Tributary Area. - The drainage area tributary to Leesville Pond on Kettle Brook is 31.3 square miles, and consists generally of rugged, wooded, sparsely settled terrain (see Plate No. 2). Many small natural ponds are found within the basin and the upper reaches of Kettle Brook are developed by a series of reservoirs for water supply for the City of Worcester. Natural run-off from all branches is flashy and retarding is negligible since the water supply reservoirs are generally filled before flooding occurs.

13. Protected Area. - The purpose of the Diversion is to provide flood protection to the City of Worcester and especially to the extensive industrial and residential property located along a reach of seven miles of the Middle River and Blackstone River above the South Works Dam of the American Steel & Wire Co.

SECTION 7

DEFINITE PROJECT PLAN

14. Recommended Project Plan. - The definite project plan for the Worcester Diversion Project is shown on Plates Nos. 3 to 7 inclusive and includes the construction of an intake structure housing the gates, a 16-foot diameter concrete tunnel in earth, extending 500 feet in length from the Leesville Pond to rock in Pakachoag Hill; a 16-foot tunnel in rock approximately 4,000 feet in length through Pakachoag Hill; a 40-foot wide channel in rock 1,000 feet in length and a 50-foot wide channel in earth 9,300 feet in length to the Blackstone River at the mouth of Hull Brook. The project requires the construction of one highway bridge under U. S. Route 20, one highway bridge under Greenwood Street, (a Town road), a railroad bridge at the New York, New Haven and Hartford Railroad and minor utility relocations. Results of borings taken at the site, logs of borings, soils and geological studies are shown on the plates and discussed in detail in Appendices II and III. For further description of struc-

tures and improvements, see Section 10.

15. Design Flood. - Detailed criteria and method of computing the design flood are shown in Appendix I, Hydrology. They are summarized as follows:

The Design Flood for the Worcester Diversion is based upon the maximum storm of record in the vicinity, that of September 1938. One of the centers of the storm was over Barre, Massachusetts, only 20 miles northeast of the center of the drainage area tributary to the Worcester Diversion. This storm center was transposed to the 31.3 sq. mile area above the Diversion Intake and infiltration was allowed at the rate at which it occurred on this area during the same storm. A 6-hour unit-hydrograph derived from the record of the September 1938 flood at the Webster Street Gaging Station was applied to the resulting 10.27 inch of rainfall excess for the transposed storm to compute the hydrograph of the design flood. This hydrograph has a peak rate of discharge of 5600 c.f.s.

16. Capacity. - The capacity of the Worcester Diversion is 5600 c.f.s. when the pool in Leesville Pond is at elevation 484.5 m.s.l. This rate of discharge is the same as the peak rates of inflow to Leesville Pond for the Design Flood.

17. Floods of Record. - A stream gaging station was established by the U. S. Geological Survey at the Webster Street Bridge just below Leesville Pond Dam, in August 1923. Since that date only two damaging floods have occurred. The flood of March 1936 had two peaks 6 days apart. The first occurred on 12 March and amounted to 1340 c.f.s. and the second occurred on 18 March and amounted to 2520 c.f.s. which is the largest recorded rate of discharge. The flood of 21 September 1938 had a peak discharge of 1300 c.f.s. The volume of runoff for the second rise of March 1936 was computed at 5.81 inches and the volume of the September 1938 flood at 4.26 inches.

18. Miscellaneous. - a. Malaria Control. - The project plan provides no special feature for malaria control. No change is made in normal surface water conditions along the channel portion of the Diversion and the only affect of the project on the breeding of mosquito larvae will be the lowering of ground waters adjacent to the outlet channel.

b. Provisions for Fish and Wildlife. - The Fish and Wildlife Service has been furnished information on this project. Any necessary provisions to take care of fish and wildlife will be made although there appear to be no adverse effects on fish and wildlife due to the Diversion.

SECTION 8

DEPARTURES FROM PROJECT DOCUMENT PLAN

19. No significant departures from the Project Document Plan are proposed. Design refinements are as follows:

a. Size of tunnel. - The survey report proposed a 17-foot diameter tunnel. This size has been reduced to 16-foot diameter by reducing the length of tunnel and lowering the tunnel outlet portal.

b. Tunnel alignment. - The tunnel alignment was modified to provide a more economical location for the project as a whole.

c. Stilling Basin. - The stilling basin has been modified to take full advantage of a location in deep rock cut.

d. Intake. - An overflow weir has been added at the intake to prevent a failure due to excessive velocities which might occur with faulty operation.

SECTION 9

OTHER PLANS INVESTIGATED

20. Alternate Plans. - The type and location of the Diversion as proposed, results from several alternate studies made.

a. Operation of Existing Reservoirs. - The plan provided for operation of 12 existing reservoirs to provide flood control storage capacity of 5-1/2 inches on 31.5 sq. miles. The plan was rejected since operation of the reservoirs for flood control was incompatible with the present usage for water supply, and power conservation. The purchase of the reservoirs was not economical. For details see paragraph 41 of H.D. 624, 78th Congress 2d Session.

b. Pakachoag Dam. - The plan provided for a single flood control dam on Kettle Brook, 2.1 miles above its confluence with Tatnuck Brook. The flood control capacity would be 11,300 acre feet equivalent to 7.0 inches of runoff from the drainage area of 30 square miles. The plan was rejected as being economically inferior to the Worcester Diversion. For details see paragraph 42 (a) of H.D. 624, 78th Congress, 2d Session.

c. Alignment. - Several alignments were studied prior to the selection of the location shown in this report. The alignments were predicated on the location of the outlet portal. After thorough consideration, the most economical, feasible location was chosen. The survey report alignment was discarded when a saving of approximately $150,000 became apparent.

d. Intake. - An alternate intake consisting of a riprapped chute was considered and discarded when it became apparent that a velocity of approximately 20 feet per second could occur over the riprap. This condition would develop if the diversion remained in operation after the flood peak passed and the chute was permitted to become the control.

e. Cut and cover conduit. - The tunnel in earth has been substituted for the cut and cover conduit shown in the survey report. The substitution was made to eliminate heavy excavation, disruption of railroad and highway traffic and to provide a suitable work area adjacent to the construction.

SECTION 10

DESCRIPTION OF PROPOSED STRUCTURE AND IMPROVEMENTS

21. Intake Weir. - A 101-foot weir is provided at the intake to prevent failures due to excessive drawdown and reduce intake maintenance. The weir is designed to pass the full capacity of the tunnel with the pond at elevation 484.5 with approach velocities not greater than 5-1/2 ft. per sec. The crest of the weir is elevation 478.0.

22. Gate structure. - The gate structure is located west of and adjacent to the New Haven Railroad along Leesville Pond. It will be a brick superstructure on a reinforced concrete substructure. The structure is founded on a suitable till more fully described in Appendix II and III.

23. Equipment. - Equipment for the Worcester Diversion will consist essentially of those items of equipment to be located in the intake structure. Equipment will include three 6' x 12', cast iron, slide type service gates operated through rigid gate stems by means of electrically-operated, screw type hoists; one structural steel emergency bulkhead capable of closing under free flow; one traveling crane to handle the emergency bulkhead and to facilitate general maintenance; a gasoline-electric standby unit to provide power in case of failure of the commercial supply; a switchboard to control light and power circuits within the intake structure; and a heating system of sufficient capacity to prevent condensation of moisture on equipment. The gate operating equipment will be arranged to limit the maximum speed of opening to a point where the initial surge of water through the tunnel and channel will be within safe limits.

24. Tunnel in earth. - The project plan includes the construction of approximately 500 feet of tunnel in earth of inside diameter of 16 feet with 20 inch thickness of reinforced concrete lining. The invert elevation at the intake is 455.0 m.s.l. which is 29.5 feet below the normal water surface (484.5 m.s.l.) of Leesville Pond. (See Plate No. 4.)

25. Tunnel in rock. - The tunnel in rock, 4000 feet in length, will be constructed through Pakachoag Hill. The tunnel will be 16 feet inside diameter with 16 inch thickness of non-reinforced concrete lining. The surrounding rock will be thoroughly grouted. The elevation for the invert of the tunnel at the outlet portal will be 407.0 m.s.l. (See Plate No. 4.)

26. Stilling basin. - A concrete lined stilling basin in rock at the outlet portal is provided to reduce the tunnel velocities from approximately 28 to 11 feet per second for the protection of the channel in earth and adjacent lands.

27. Channel in rock. - A channel will be constructed in rock from the outlet portal to a point approximately 1000 feet downstream. The cross-section of the channel will be 40 feet wide with side slopes of one horizontal to four vertical. The maximum velocity in the channel is 11 feet per second. The invert elevation of the channel at the stilling basin end weir will be 407.0 m.s.l. and the slope will be approximately 0.001. A 3' x 2' pilot channel will be cut in the bottom to drain the

stilling basin. Material not used as riprap will be spoiled along the banks, covered with spoil from the earth channel and grassed. (See Plate No. 5.)

28. Channel in earth. - A channel in earth will be constructed from the end of the channel in rock approximately 9300 feet to the Blackstone River. The elevation of the invert of the channel at the Blackstone River is 397.0 ± m.s.l. The cross-section of the channel will be 50 feet wide with side slopes of one vertical to two and one-half horizontal. The slope of the channel is 0.001. The maximum velocity in the earth channel is 6 feet per second. The bridge abutments are to be protected by hand placed riprap and a concrete lining in the bottom of channel under Route No. 20 highway bridge. The channel between station 54+00± and 112+00± is protected by 2 feet of graded tunnel muck on the tangents and 2 feet of dumped rock on 9 inches of bedding at the curves. This proposed protection is based on the economic use of materials from tunnel and channel excavation and is considered to provide reasonable assurance of adequate protection. More extensive use of hand placed riprap would greatly increase the cost of the project and would be of questionable justification in view of the present relatively undeveloped condition of lands immediately adjacent to the channel. A pilot channel of varying cross-section will be provided in the bottom to carry the normal flow of Hull Brook and its tributaries. Spoil will be neatly piled along the edges of the channel in suitable locations and grassed. The location of spoil piles shown on the plan is tentative and subject to revision in subsequent agreement with the City of Worcester. (See Plates Nos. 5 and 6.)

SECTION 11

RECREATIONAL DEVELOPMENT

29. Recreation facilities. - The project is not suitable for the development of recreational facilities. The project area will be neatly grassed and left in a condition similar in general landscape appearance to the grounds of the nearby Worcester Sewage Treatment Plant.

SECTION 12-A

RELOCATIONS

30. Facilities Affected. - The construction of the project will necessitate the following changes to existing facilities. For details see Appendix VII-A, Relocations.

a. Highways. - No highway relocations will be required except a minor revision of grade at the Greenwood Street bridge.

b. Highway Bridges. - Two new bridges of 50-foot clear span will be required where the channel crosses U.S. Route #20 and at Greenwood Street. For major dimensions of structures, see Plates Nos. 5 and 6.

c. Railroad Bridge. - A new double track railroad bridge of 50-foot clear span will be required where the channel crosses the New York, New Haven and Hartford Railroad near the Blackstone River. For major structure dimensions see Plate No. 6.

d. Electric Power Distribution and Telephone Lines. - Two or three electric and telephone service poles will be relocated to avoid interference with the proposed project.

e. Petroleum Pipe Line. - The 6-inch steel petroleum pipe line owned by the Socony-Vacuum Company will be lowered on its present location to a new grade below the channel bottom.

f. Water Main. - The 8" water main in Greenwood Street will be relocated as a part of the Greenwood Street Bridge construction.

31. Methods of Accomplishment. - It is contemplated that the relocations will be accomplished by agreements between the City of Worcester and the owners of the facilities. The cost of the relocations will be borne by local interests in accordance with the recommendations by the Chief of Engineers in House Document No. 624, 78th Congress, 2d Session.

32. Cost. - The estimated total cost of relocations is $245,000. For details, see Appendix VII-A, Relocations.

SECTION 12-B

REAL ESTATE PLANNING

33. General. - Real estate planning for this project is based upon acquisition of all rights-of-way and lands by the City of Worcester in accordance with the recommendations of the Chief of Engineers (H. Doc. No. 624, 78th Congress, 2d Session). In general, fee simple title will probably be acquired to lands in the intake structure area, outlet portal area and channel area. Easements and/or permits should suffice for the areas under which the tunnel will be constructed. For further discussion regarding the recommended character of acquisition, see Appendix VII-B. The estimated value of the lands and improvements were determined after reconnaissance of the area involved.

34. Real Estate Planning. - The estimated total cost of Real Estate is $76,000. Extended discussions, recommendations, and estimates of cost for the acquisition and easement of property situated in the project area will be found in Appendix VII-B, Real Estate Planning.

SECTION 13

AVAILABILITY OF CONSTRUCTION MATERIALS

35. Construction materials are available at the site of the work or in close proximity thereto. Concrete is available at a ready-mix plant located one-half mile from the intake. Riprap and dumped rock fill will be obtained from open cut rock excavation. Owing to a scarcity of suitable gravel in the area, riprap bedding will be obtained from tunnel muck. Processed tunnel muck will also be utilized for slope protection on earth cut channel sections. Topsoil will be obtained from stripping.

TABLE 1

CONSTRUCTION SEQUENCE AND FISCAL YEAR FUND SCHEDULE

CONSTRUCTION YEAR	FIRST												SECOND											
Fiscal Year	FIRST						SECOND												THIRD					
Month	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D
Clearing $2,200				0		100																		
Water Control $27,500				0		60			100															
Earth Excavation $292,000						0 20						100												
Earth Excavation (Struct.) $ 13,700					0	50								50		100								
Earth Excavation (Tunnel) $ 37,300					0	30				100														
Rock Excavation (Open) $242,000				0		95						100												
Rock Excavation (Tunnel) $623,700					0	20									100									
Backfill $2,600																			0	100				
Topsoil $24,800								0		40						40		80			100			
Fertilizing and Seeding $23,100								0		40						40		80			100			
Dumped Rock (24") $49,500								0									100							
Riprap, Hand Placed (12") $61,500																0	100							
Graded Tunnel Muck (24") $55,000								0									100							
Bedding $35,600								0									100							
Tunnel Grouting $220,000																	0	20					100	
Tunnel Liner Plates $ 75,900					0	20				100														
Concrete (Plain) $ 73,900																				0		100		
Concrete (Reinforced) $103,700														0				90	100					
Concrete Tunnel Lining (in earth) $61,000										0				100										
Concrete Tunnel Lining (in rock) $660,000																0		80	100					
Gates and Machinery $55,000																0		30			100			
Gatehouse Superstructure $55,000																				0			100	
Engineering & Overhead $558,000 *			24			30												80						100
Project Total $3,353,000			0.5			18												82						100
FUNDS EXPENDED BY **FISCAL** YEAR	1st Fiscal Year $594,000 *						2nd Fiscal Year $2,107,800												3rd Fiscal Year $651,200					

NOTE: Figures under bar scale indicate estimated completion in percent.
* Includes $133,000 estimated total expenditure prior to 1 April 1948, for Definite Project Report, Plans and Specifications, and overhead.

SECTION 14

SCHEDULES FOR PLANNING AND CONSTRUCTION

36. General. - The estimated time for completing Worcester Diversion and appurtenant features is 20 months. The work, exclusive of facilities involved in relocations to be accomplished by local interests, will be performed under one principal contract.

37. Contract for Worcester Diversion. - Assuming that tunneling operations commence at the outlet portal and proceed towards the inlet portal simultaneously with tunnel operations in earth from the upper heading, 14 months will be required to complete tunneling and the diversion channel. An additional period of 6 months will be required to complete the gate structure, tunnel lining, and the intake and outlet structures. Construction of the channel bridges (by the City) and other features of the work will be completed during the first year of tunneling. In planning the tunneling operations, work from the outlet portal heading has been considered to be a prime operation; holing through should take place at a point only a short distance below the junction of the tunnel in earth and tunnel in rock. This condition has been assumed because of the slope of the tunnel which decreases the pumping requirements and the fact that tunneling in earth will be relatively slow. Table I (opposite page) indicates the major items of work involved, the estimated cost and construction period of each.

38. Funds required by Fiscal Year:

First Fiscal Year:	
Contract for Construction (incl. Eng. and Overhead)	$594,000*
Second Fiscal Year:	
Contract for Construction (incl. Eng. and Overhead)	$2,107,800
Third Fiscal Year:	
Contract for Construction (incl. Eng. and Overhead)	651,200
Total	$3,353,000

It is estimated that a minimum of 3 months will be required to prepare contract plans and specifications after the approval of the Definite Project Report and receipt of assurances of local cooperation (see Section 19). The estimated cost of preparing the contract drawings and specifications is $61,000 (including overhead).

*Includes $133,000 estimated total expenditure prior to 1 April 1948, for Definite Project Report, Plans and Specifications, and overhead.

39. Man Hours of Employment. - Man hour employment on this project is estimated at 1,000,000 man hours, including onsite and offsite employment. This estimate is based on 24-hour operation within the tunnel. Table II following shows the breakdown of man hours as estimated for each classification of labor, and appurtenant data.

TABLE II

Expenditure for:-	
Labor at site	$1,473,000
Material and Plant	1,480,000
Other Expense and Profit	400,000
Construction Cost	$3,353,000
Man hours created, total	1,000,000
At construction site	600,000
Processing, transportation and administration	400,000
Value of materials, total	304,000
Cement, stone, clay and glass products	200,000
Iron and steel products	30,000
Machinery (except electrical)	20,000
Electrical apparatus	6,000
Forest products	10,000
Chemicals and paint	8,000
Other miscellaneous material	30,000
Labor Classification:	
Supervisory, man hours	50,000
Skilled, man hours	400,000
Unskilled, man hours	550,000

It is estimated that an average of 180 men would be employed at the construction site for 20 months based upon a 40-hour week.

SECTION 15

OPERATION AND MAINTENANCE

40. General. - The Worcester Diversion will be operated and maintained by the City of Worcester, in accordance with rules as prescribed by the Secretary of War.

41. Operation of the Diversion. - The Flood Control operation of the Diversion will be carried out to reduce flood damages in the City of Worcester. The major effect of the Diversion will be to reduce flood damages along Middle River with lesser effects being felt in the vicinity of the American Steel & Wire, South Works dam located 1 mile upstream of

the outlet. To this end, the gates of the Diversion will be operated whenever necessary to maintain the elevation of Leesville Pond at 484.5 m.s.l. thereby preventing the passage of flood flows over the Leesville Pond spillway. Reference is made to paragraph 4-03 of Appendix IV for additional details of the operation of the Diversion.

42. Maintenance. - Periodic inspection of the Diversion and appurtenant structures and equipment will be made by the Division Engineer, New England Division, Boston, Massachusetts. Regular care and inspection will be the duty of the operator who will be provided by the City of Worcester.

43. Annual Cost. - It is estimated that the total annual cost, including operation and maintenance, of the Worcester Diversion will approximate $167,500 per year. Annual cost to local interests will approximate $17,700 per year.

SECTION 16

MALARIA CONTROL

44. Malaria Control.- The project plan provides no special features for malaria control since no change is made in the normal surface water conditions in Leesville Pond and the channel portion of the Diversion, and is designed to provide drainage of seepage and surface waters intercepted. It is probable that the project will have some beneficial effect in reducing the breeding of mosquito larvae by reason of the lowering of ground waters that now create some swampy areas adjacent to the outlet channel.

SECTION 17

COST ESTIMATES

45. Allocation of Cost of Project. - The estimated grand total capital cost of this project is $3,674,000, which is all allocated for Flood Control. The cost to the United States will be $3,353,000. The cost to local interests will be $321,000.

46. Total Cost. - The total estimated cost (1946 prices) of $3,674,000 is made up of the items as listed below:

Item	Quantity	Unit	Unit Price	Amount
1 CONSTRUCTION				
Clearing			L.S.	$ 2,000
Stream Control			L.S.	25,000
Excavation, Earth (common)	590,000	C.Y.	0.45	265,500
Excavation, Earth (structural)	6,200	C.Y.	2.00	12,400
Excavation, Earth (tunnel)	5,650	C.Y.	6.00	33,900
Excavation, Rock (open cut)	80,000	C.Y.	2.75	220,000
Excavation, Rock (tunnel)	40,500	C.Y.	14.00	567,000
Backfill	4,800	C.Y.	.50	2,400
Dumped Rock	18,000	C.Y.	2.50	45,000
Graded Tunnel Muck	20,000	C.Y.	2.50	50,000

Item	Quantity	Unit	Unit Price	Amount
Riprap, Hand Placed (12")	8,000	C.Y.	7.00	56,000
Bedding (tunnel muck)	10,800	C.Y.	3.00	32,400
Tunnel Grouting			L.S.	200,000
Tunnel Liner, Plates	460,000	Lb.	.15	69,000
Concrete, Plain	3,730	C.Y.	18.00	67,140
Concrete, Reinforced	2,355	C.Y.	40.00	94,200
Concrete Tunnel Lining (in earth)	1,790	C.Y.	31.00	55,490
Concrete Tunnel Lining (in rock)	20,000	C.Y.	30.00	600,000
Gates & Machinery			L.S.	50,000
Gatehouse & Miscellaneous			L.S.	50,000
Topsoil	18,000	C.Y.	1.25	22,500
Fertilizing & Seeding	70	Ac.	300.00	21,000
Sub-total				$2,541,000
Contingencies 10%				254,000
Total Construction Cost				$2,795,000
2. DEFINITE PROJECT REPORT				65,000
3. PLANS AND SPECIFICATIONS				54,000
4. SUPERVISION & INSPECTION CONSTRUCTION				221,000
5. COMPLETION REPORT				8,000
6. DIVISION & DISTRICT OVERHEAD				210,000
Total Estimated Government Cost				$3,353,000

Item	Quantity	Unit	Unit Price	Amount
7. RELOCATIONS (Estimated total cost including bridges, care of water main, petroleum pipeline, and engineering and overhead.) (See Appendix VII-A)				245,000
8. LANDS & RIGHTS-OF-WAY (Estimated total cost including land, buildings, rights-of-way, legal and overhead.) (See Appendix VII-B)				76,000
Total Estimated Cost to Local Interests				321,000
GRAND TOTAL COST OF PROJECT				$3,674,000

SECTION 18

BENEFITS

47. Benefits. - The Worcester Diversion will provide protection against all damages in the City of Worcester due to a repetition of the flood of record (March 1936). Benefits for the design flood vary from complete protection at and above the confluence of Kettle and Tatnuck Brook to 90% (basis of damages prevented on 1939 valuations) at the south works of the American Steel and Wire Company. The following table indicates the amount of the losses prevented:

Location of Damage Zone	1936 Flood Nat.	1936 Flood Mod.	Design Flood Natural	Design Flood Modified
Diversion to Curtis Pond Dam	$ 48,600	0	$ 216,000	0
Curtis Pond Dam to Middle River Bridge	339,000	0	1,740,000	$180,000
Middle River Bridge to End Diversion	320,700	0	2,320,000	190,000

(The above figures are based on 1939 valuations.)

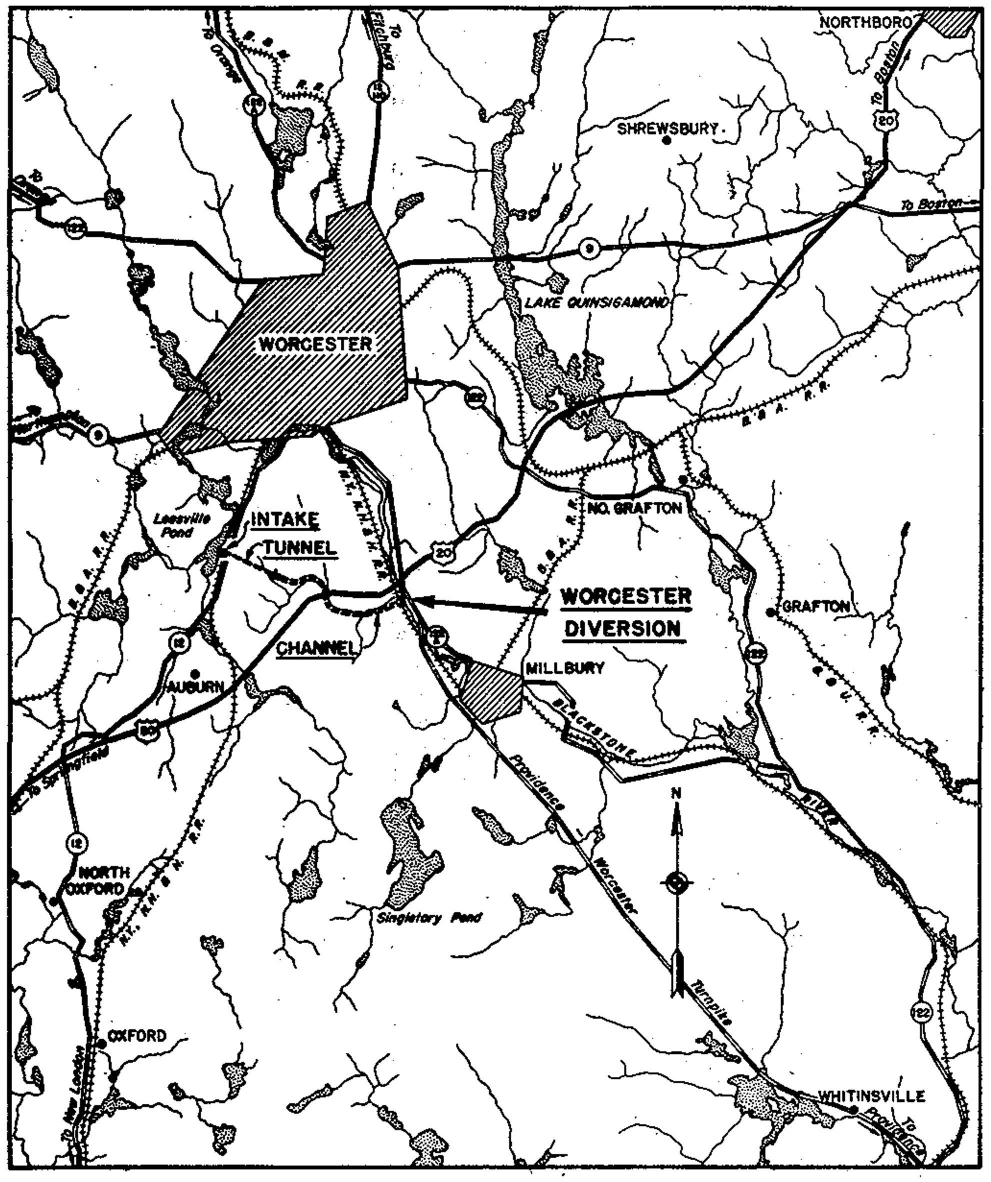
The area benefitted consists of highly developed industrial and residential developments, including approximately 80 homes, 40 stores, 22 manufacturing plants, which in the maximum flood of record (March 1936) suffered total direct damages of $708,300, with indirect damages approximately 50 percent as great.

SECTION 19

RECOMMENDATION

48. I recommend that the Worcester Diversion flood control project be constructed in general accordance with the descriptions set forth in this Definite Project Report.

D. L. WEART
Brigadier General, U. S. A.
Division Engineer

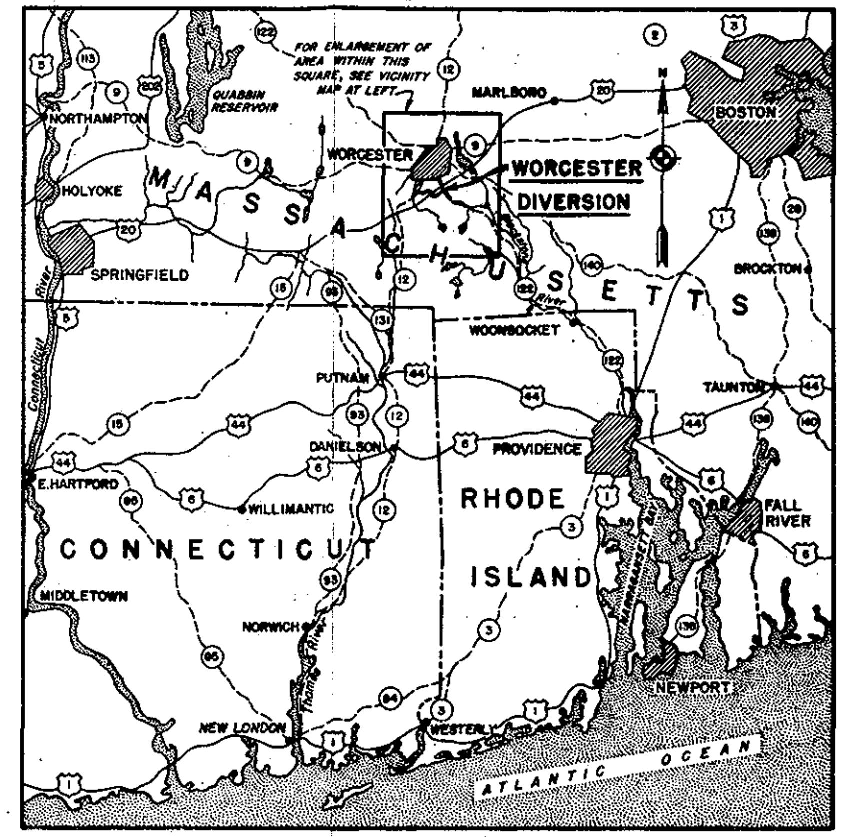


VICINITY MAP

SCALE $\frac{1}{62,500}$

LEGEND

- U.S. highways
- State highways
- Railroads
- Rivers



LOCATION MAP

SCALE 1 IN. = 8 MI.

LEGEND

- U.S. highways
- State highways
- State lines
- Rivers

KEY	DATE	REVISION (Indicated by △)	REV.BY	CK.BY	AP.BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

VICINITY AND LOCATION MAPS

BLACKSTONE RIVER — MASSACHUSETTS

IN SHEETS — SCALE 1:62,500 — SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: SENIOR ENGINEER, HEAD, PLANNING BRANCH

APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV.

APPROVED: CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: PLANNING BRANCH

DRAWN: D.C. O'R.
TRACED: D.H.R.
CHECKED: D.P.

FILE NO. BE-1-1016

PLATE NO. 1

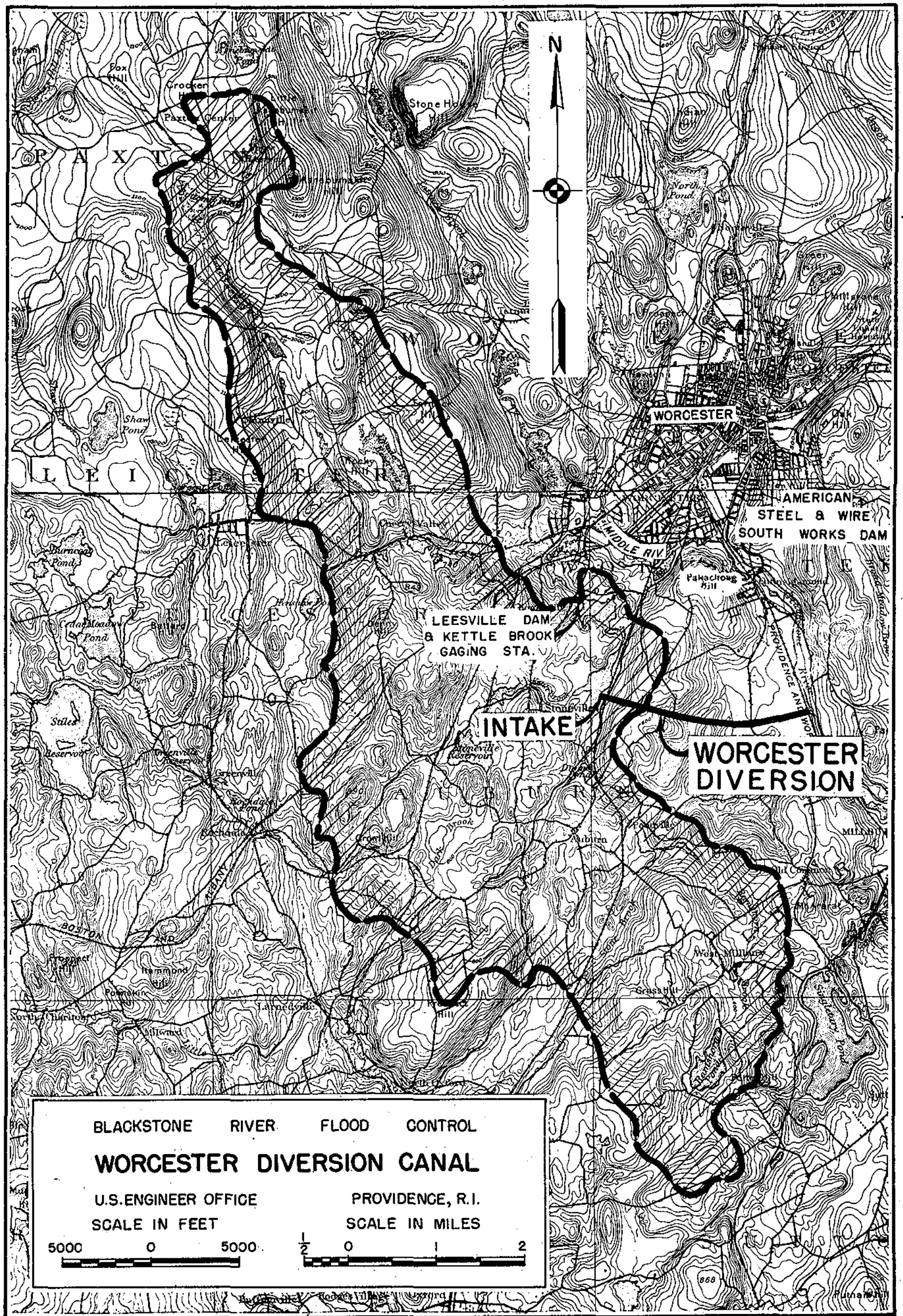
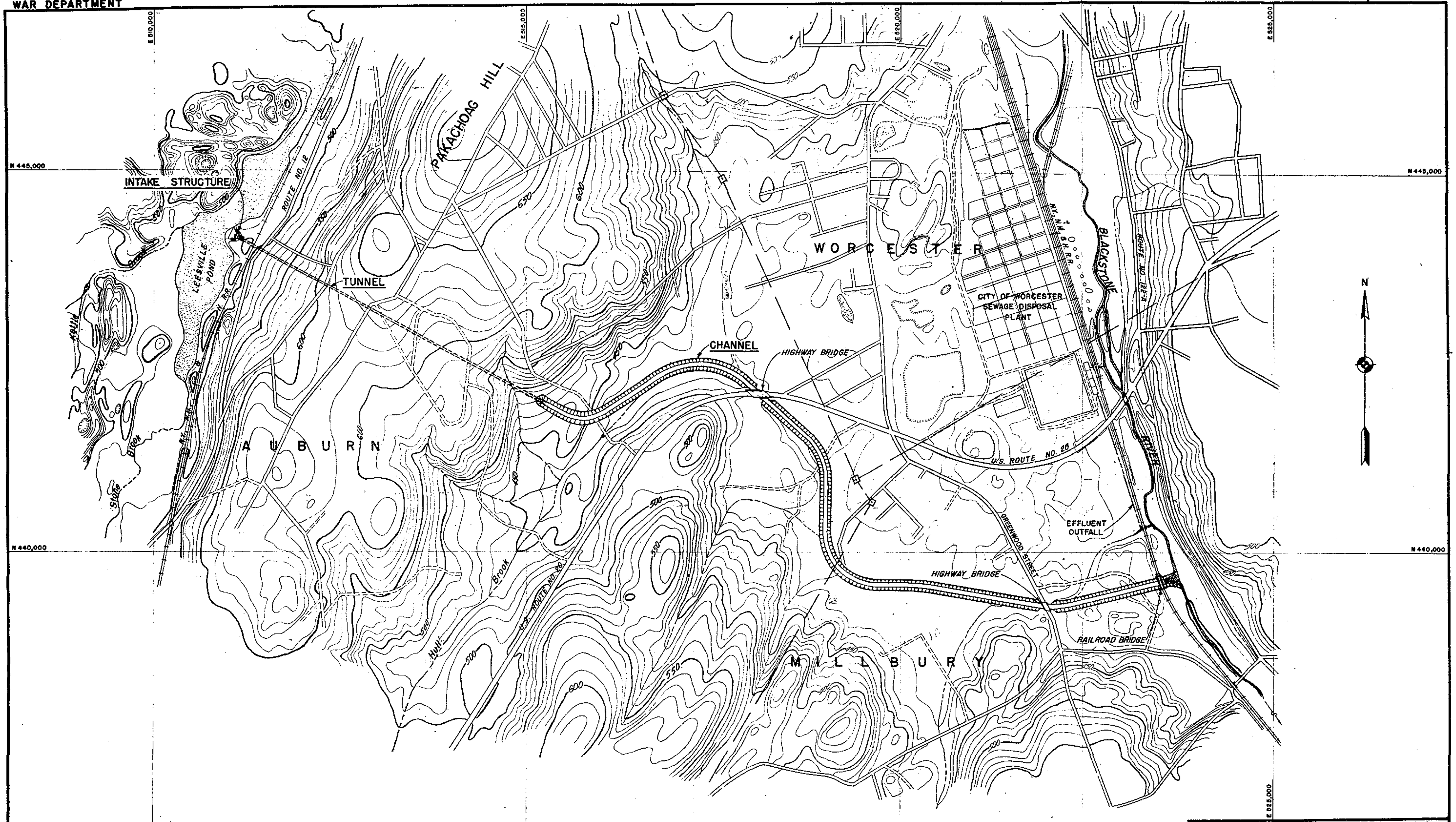

N
WORCESTER
AMERICAN
STEEL & WIRE
SOUTH WORKS DAM
MIDDLE RIV.
LEESVILLE DAM
& KETTLE BROOK
GAGING STA.
INTAKE
WORCESTER
DIVERSION
PROVIDENCE AND WORCESTER
BLACKSTONE RIVER FLOOD CONTROL
WORCESTER DIVERSION CANAL
U.S. ENGINEER OFFICE
PROVIDENCE, R.I.
SCALE IN FEET
5000 0 5000
SCALE IN MILES
1/2 0 1 2

PLATE NO. 2

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTE

Topography taken from U.S.G.S. quadrangle sheets.
Bridges to be provided by local interests.

KEY	DATE	REVISION (indicated by △)	REV.BY	CK.BY	AP.BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

GENERAL PLAN

BLACKSTONE RIVER — MASSACHUSETTS

IN SHEETS — SCALE: 1 IN. = 500 FT. — SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JULY 1946

SUBMITTED: SENIOR ENGINEER, HEAD, PLANNING BRANCH
APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV.
APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

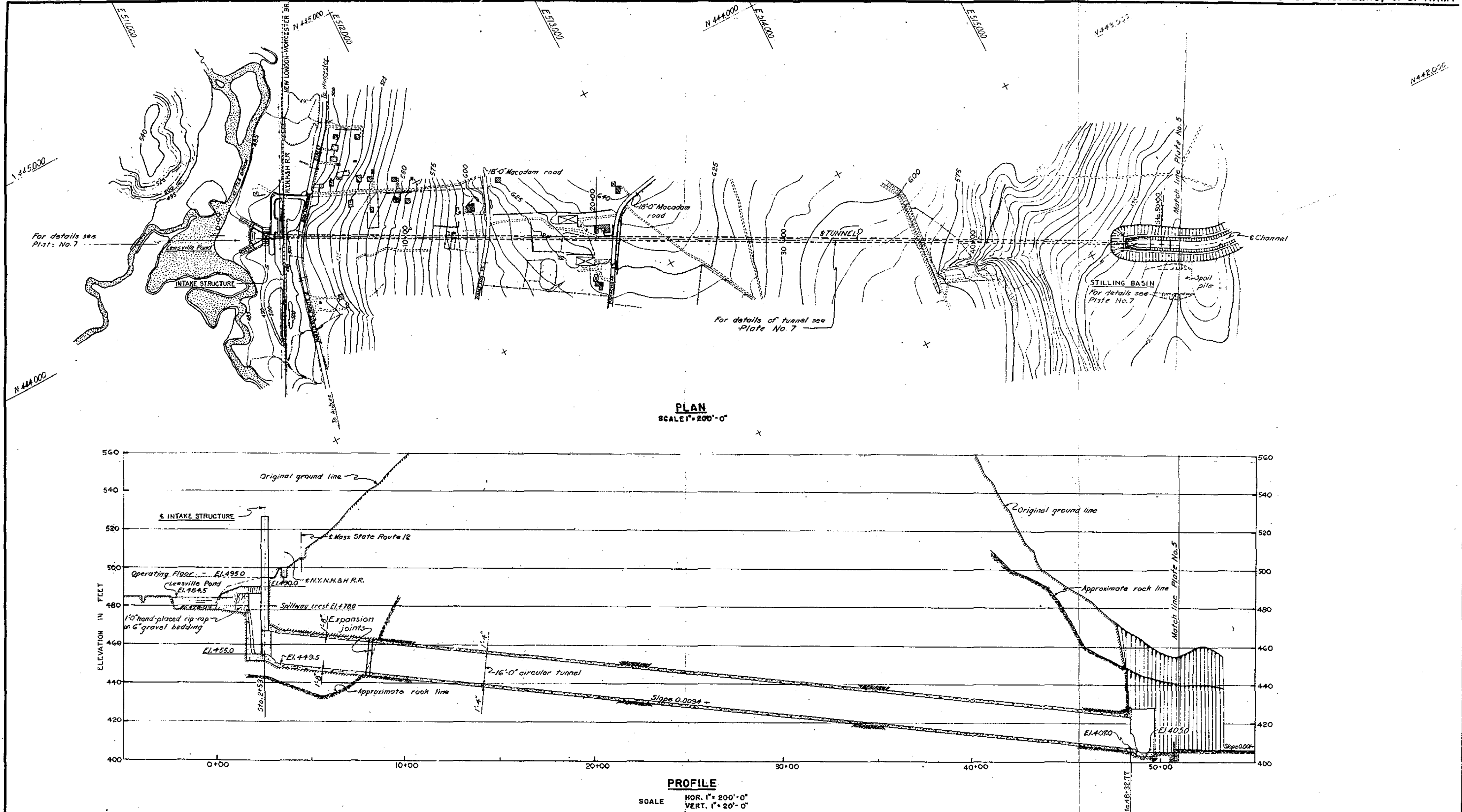
PREPARED: PLANNING BRANCH — DRAWN: — TRACED: — CHECKED:

FILE NO. BE-1-1017

PLATE NO. 3

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTES

Elevations refer to Mean Sea Level Datum.
Contour interval is 5 feet.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

PLAN AND PROFILE NO. 1

BLACKSTONE RIVER MASSACHUSETTS

IN SHEETS SCALE 1 IN. = 200 FT. SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: SENIOR ENGINEER HEAD, PLANNING BRANCH
APPROVAL RECOMMENDED: HEAD ENGINEER CHIEF, ENGINEERING DIV.
APPROVED: COL. CORPS OF ENGINEERS DISTRICT ENGINEER

PREPARED: PLANNING BRANCH
DRAWN: J.S.M.
TRACED:
CHECKED:

FILE NO. BE-1-1018

PLATE NO. 4

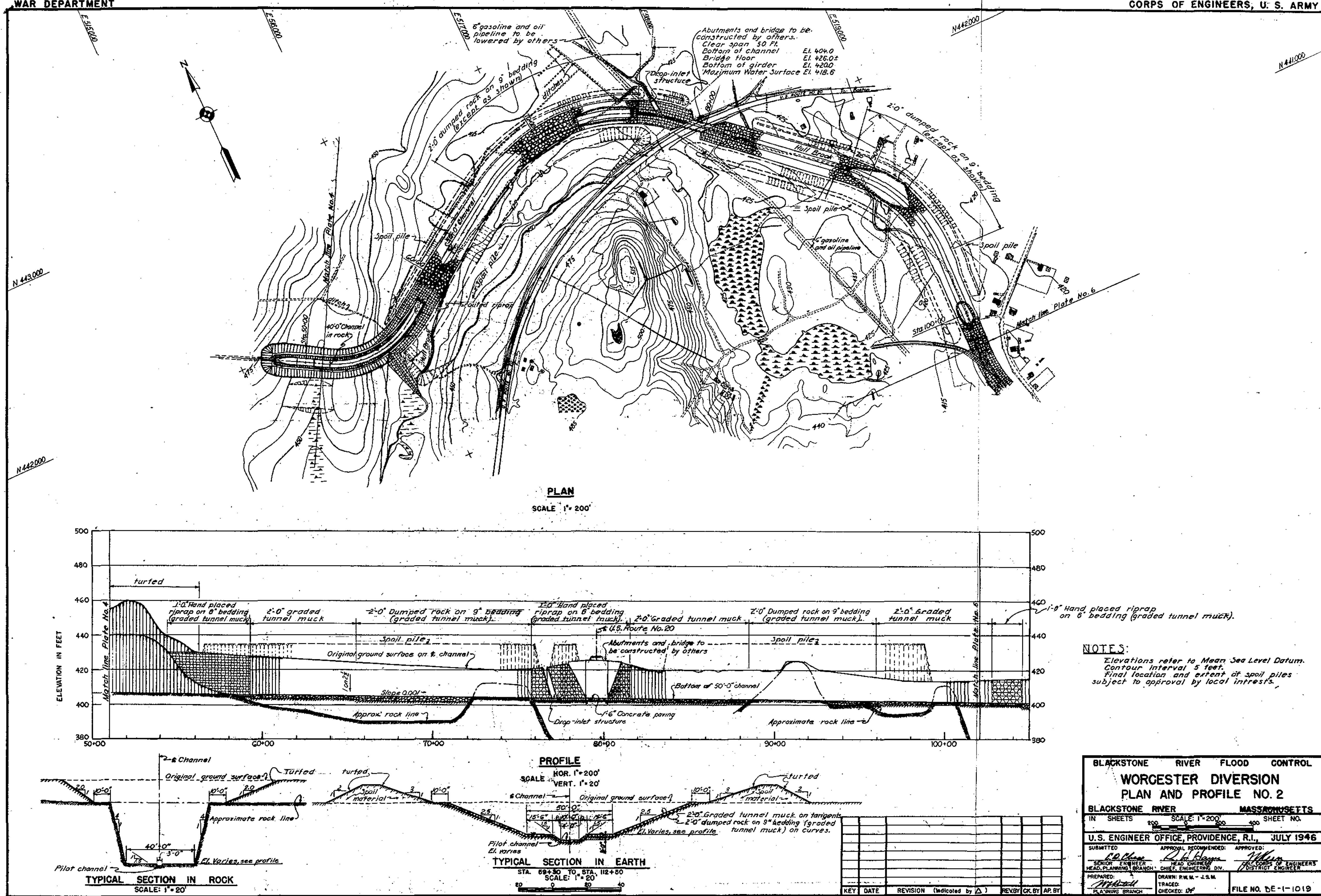
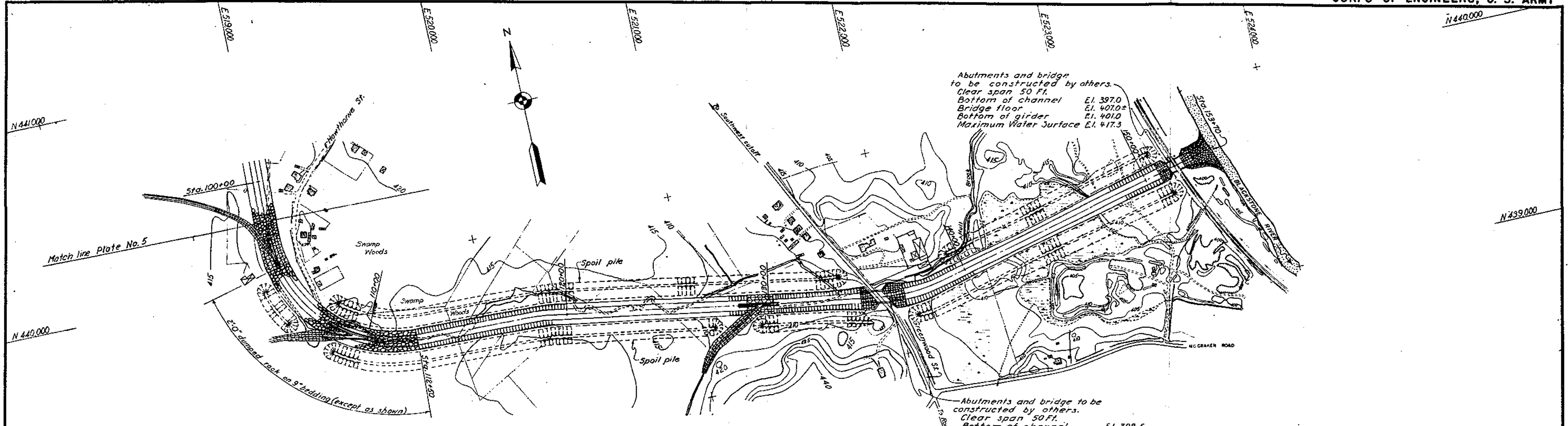

WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
6" gasoline and oil pipeline to be lowered by others
Abutments and bridge to be constructed by others.
Clear span 50 Ft.
Bottom of channel El. 404.0
Bridge floor El. 426.0±
Bottom of girder El. 420.0
Maximum Water Surface El. 418.6
Drop-inlet structure
2'-0" dumped rock on 9" bedding (except as shown)
Hull Brook
Spoil pile
6" gasoline and oil pipeline
Grouted riprap
40'-0" Channel in rock
Match line Plate No. 4
Match line Plate No. 6
PLAN
SCALE 1" = 200'
ELEVATION IN FEET
turfed
1'-0" Hand placed riprap on 6" bedding (graded tunnel muck)
2'-0" graded tunnel muck
2'-0" Dumped rock on 9" bedding (graded tunnel muck)
1'-0" Hand placed riprap on 6" bedding (graded tunnel muck)
2'-0" Graded tunnel muck
2'-0" Dumped rock on 9" bedding (graded tunnel muck)
2'-0" Graded tunnel muck
1'-0" Hand placed riprap on 6" bedding (graded tunnel muck).
℄ U.S. Route No. 20
Abutments and bridge to be constructed by others
Original ground surface on ℄ channel
Slope 0.001
Bottom of 50'-0" channel
Approx. rock line
1'-6" Concrete paving
Drop-inlet structure
Approximate rock line
NOTES:
Elevations refer to Mean Sea Level Datum.
Contour interval 5 feet.
Final location and extent of spoil piles subject to approval by local intrests.
PROFILE
SCALE HOR. 1" = 200' VERT. 1" = 20'
℄ Channel
Original ground surface
Turfed
Approximate rock line
El. Varies, see profile
Pilot channel
TYPICAL SECTION IN ROCK
SCALE: 1" = 20'
Spoil material
2'-0" Graded tunnel muck on tangents
2'-0" dumped rock on 9" bedding (graded tunnel muck) on curves.
El. Varies, see profile
Pilot channel El. varies
TYPICAL SECTION IN EARTH
STA. 69+50 TO STA. 112+50
SCALE: 1" = 20'
KEY DATE REVISION (Indicated by △) REV. BY CK. BY AP. BY
BLACKSTONE RIVER FLOOD CONTROL
WORCESTER DIVERSION
PLAN AND PROFILE NO. 2
BLACKSTONE RIVER MASSACHUSETTS
IN SHEETS SCALE: 1" = 200' SHEET NO.
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946
SUBMITTED
SENIOR ENGINEER HEAD, PLANNING BRANCH
APPROVAL RECOMMENDED:
HEAD ENGINEER CHIEF, ENGINEERING DIV.
APPROVED:
COL., CORPS OF ENGINEERS DISTRICT ENGINEER
PREPARED:
PLANNING BRANCH
DRAWN: R.W.M. - J.S.M.
TRACED:
CHECKED:
FILE NO. BE-1-1019

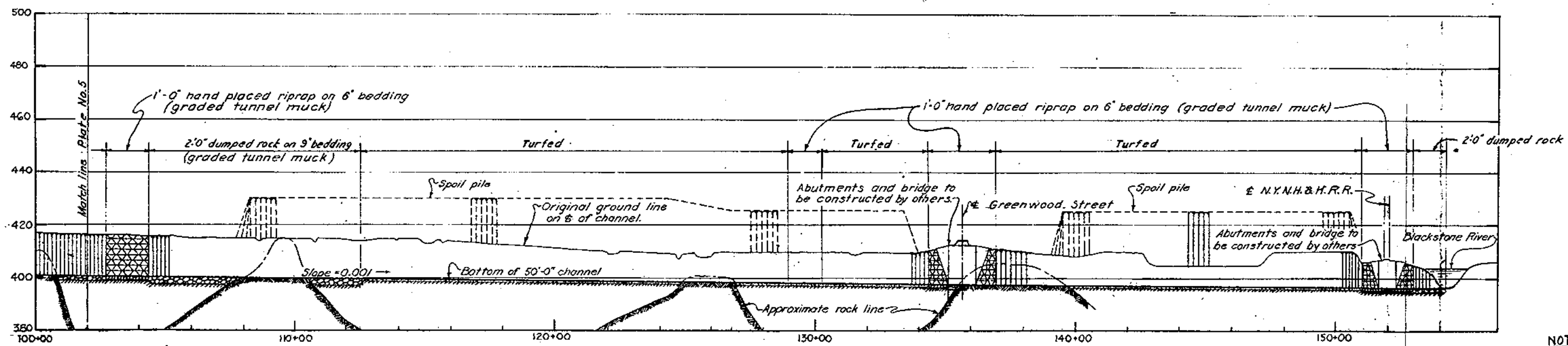
PLATE NO. 5

WAR DEPARTMENT

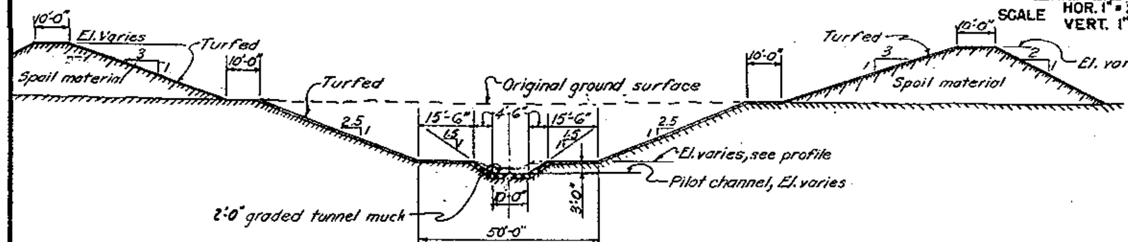
CORPS OF ENGINEERS, U. S. ARMY



PLAN
SCALE 1" = 200'



PROFILE
SCALE HOR. 1" = 200' VERT. 1" = 20'



TYPICAL SECTION
STA. 112+50 — 134+60
STA. 136+75 — 150+70
SCALE 1" = 20'

NOTES

Elevations refer to Mean Sea Level Datum.
Contour interval 5 feet.
Channel slopes, except where protected with dumped rock, riprap or tunnel muck, and surfaces of spoil piles will be mulched and seeded. Where exposed material is unsuitable to support growth a layer of topsoil not less than 6 inches on channel slopes and 3 inches on spoil piles will be placed before mulching and seeding.
Final location and extent of spoil piles subject to approval by local interest.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

PLAN AND PROFILE NO. 3

BLACKSTONE RIVER — MASSACHUSETTS

IN SHEETS — SCALE 1 IN. = 200 FT. — SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: SENIOR ENGINEER, HEAD, PLANNING BRANCH — APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV. — APPROVED: COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: PLANNING BRANCH — DRAWN: P.W.M. — TRACED: — CHECKED: — FILE NO. BE-1-1020

PLATE NO. 6

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

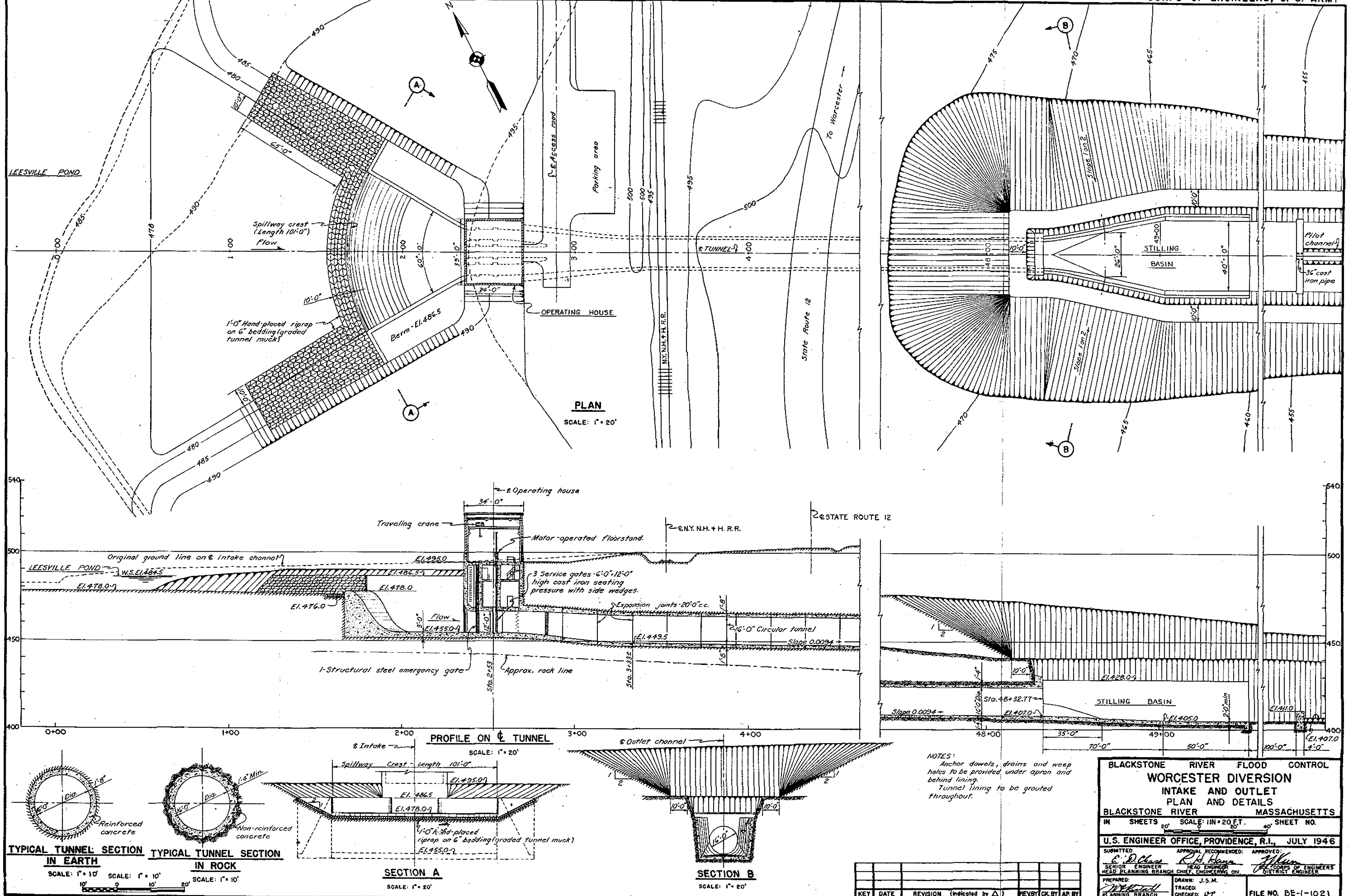


PLATE NO. 7

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX I

HYDROLOGY

To Accompany Definite Project Report
dated Sept. 1946

WORCESTER DIVERSION

APPENDIX I. HYDROLOGY

CONTENTS

WORCESTER DIVERSION

TABLES

WORCESTER DIVERSION

PLATES

APPENDIX I. HYDROLOGY

1-01. Climate. - a. Temperatures. - The mean annual temperature for 53 years of record at Worcester, Massachusetts, is 47.8 degrees F. Summer temperatures rarely rise to 100 degrees F. Freezing temperatures are to be expected between the latter part of November and the latter part of March, and temperatures may fall to 0 degrees F. or lower several times during a winter. The average monthly temperatures and the maximum and minimum recorded each month for 53 years of record at Worcester, ending with 1945, are as follows:

TABLE 1. TEMPERATURES AT WORCESTER, MASS.

Monthly average, maximum and minimum for 53 years of record

Month	Average °F	Maximum °F	Minimum °F	Month	Average °F	Maximum °F	Minimum °F
Jan.	25.1	67	-17	July	70.7	102	41
Feb.	25.1	65	-24	Aug.	68.7	99	38
Mar.	34.7	84	- 6	Sep.	61.9	96	28
Apr.	45.4	89	9	Oct.	51.2	89	15
May	57.2	92	27	Nov.	39.7	75	3
June	65.9	96	33	Dec.	27.9	67	-17

b. Precipitation. - (1) Average Rainfall. - The mean annual precipitation at the U. S. Weather Bureau station in Worcester, Massachusetts, is 42.76 inches for 81 years of record extending, with a few interruptions, from January 1853 to December 1945, inclusive. The maximum annual amount for this same station and period was 61.71 inches in 1888 and the minimum 30.96 inches in 1914. Table 2, which follows, summarizes the precipitation record at Worcester by calendar months.

TABLE 2. MONTHLY PRECIPITATION AT WORCESTER, MASS.

	Depth in Inches				Depth in Inches		
Month	Mean	Maximum	Minimum	Month	Mean	Maximum	Minimum
Jan.	3.51	9.03	1.02	July	3.64	11.41	0.62
Feb.	3.16	8.09	0.67	Aug.	3.90	13.14	0.35
Mar.	3.82	11.13	0.04	Sep.	3.54	10.82	0.20
Apr.	3.51	8.87	0.51	Oct.	3.57	9.81	0.36
May	3.61	8.84	0.88	Nov.	3.66	9.82	0.56
June	3.38	8.31	0.66	Dec.	3.46	7.77	0.84

All rainfall stations in the vicinity of Worcester are listed on Plate No. I-1 with their respective periods of record and are located on a map of the area on Plate No. I-2.

(2) Storms. - Three types of storms are characteristic of the region in which the Worcester Diversion project is located, viz; (1) continental storms, (2) hurricanes and (3) thunderstorms. Continental storms may be of the stationary frontal type or rapidly moving intense cyclones; they are not limited to any season or month but follow one another at more or less regular intervals and with varying intensities throughout the year. The usual path of hurricanes lies to the south and east of New England but they may be deflected over this area by continental cyclonic disturbances. They are most likely to occur during the summer and autumn months. Thunderstorms may be of local origin, or they may be of the frontal type associated with continental storms during the summer months.

A definite combination of meteorological conditions is recognized as being responsible for most of the great flood-producing storms of the northeastern United States. They are (1) a persistent high-pressure area over the western North Atlantic Ocean, (2) another high-pressure area over the central and northern interior of the continent and (3) a low-pressure trough between these "highs" including one or more moving centers over the northeastern states.

The storm of September 1938, which is the most severe of record for New England and for the area under study herein, was of this type. During this storm about 8.2 inches of rain fell on the drainage area of the Blackstone River above Worcester, Mass., from 1 P.M. on 18 September to 6 P.M. on 21 September. This storm had two centers of maximum precipitation, one of over 17 inches depth centering over East Glastonbury, Conn., and the other of over 16 inches depth centering over Barre, Mass. This latter center was only 19 miles from Worcester and it was the most severe storm ever recorded in this vicinity. The locations of the centers of this and other outstanding storms of record in the northeastern United States are shown on Plate No. I-3, together with their area-depth characteristics.

(3) Snowfall and Snow Cover. - The average monthly and annual snowfall at Worcester for 53 years of record is shown in the following table:

TABLE 3. AVERAGE MONTHLY AND ANNUAL SNOWFALL AT WORCESTER, MASSACHUSETTS

Month	Depth, inches	Month	Depth, inches
January	14.5	July	-
February	16.1	August	-
March	9.1	September	-
April	3.1	October	0.1
May	-	November	3.3
June	-	December	9.9
Average annual snowfall 56.1 inches			

Snow cover, on the average, is at a maximum about 15 February. Heavy spring runoff resulting from the melting of snow cover occurs almost every year, but there is no record that this factor alone ever caused a damaging flood. To contribute to a major flood the runoff from melting snow must be augmented by heavy rainfall. All available records indicate that for a drainage area of the size involved in this project the maximum rate of runoff from a combination of melting snow and rain will be less than can result from the more intense rates of rainfall possible during the summer and autumn months.

1-02. Runoff. - a. Records. - A water-stage recorder has been located on the Blackstone River (also known as Kettle Brook at this point) at Webster Street Bridge, Worcester, since August 1923. The drainage area above this gage is 31.3 square miles and the gage is just below the Leesville Pond Dam. Hence 21 years of runoff records are available for the area tributary to the proposed diversion tunnel. The mean, maximum and minimum annual and monthly runoff as measured at this station through September 1944 are shown in the following table:

TABLE 4. SUMMARY OF RUNOFF - BLACKSTONE RIVER AT WORCESTER, MASS.

(Drainage area of U.S.G.S. gaging station = 31.3 square miles)

Period	Runoff in inches			Mean runoff as
	Mean	Maximum	Minimum	percent of rainfall
January	2.06	4.53	0.46	57.5
February	1.82	3.06	0.89	62.8
March	3.75	11.53	1.71	94.2
April	3.54	7.42	1.21	97.0
May	2.05	4.21	0.89	64.3
June	1.43	3.85	0.62	36.9
July	1.12	6.01	0.53	30.9
August	0.85	2.12	0.35	24.1
September	1.15	5.28	0.32	28.3
October	1.02	2.52	0.27	29.8
November	1.45	4.53	0.46	37.9
December	1.83	4.42	0.35	53.2
Year	22.07	36.03	9.70	51.2

b. Floods. - (1) Historical Floods. - Information is meager regarding floods at Worcester prior to installation of the stream gaging station described in the foregoing paragraph. Floods are known to have occurred in the lower Blackstone River in March 1818, March 1876, March 1877, February 1886, and September 1887, but there is no record of severe flooding having occurred at Worcester during these floods.

(2) Floods of Record. - Since the U.S.G.S. gaging station was established at the Webster Street Bridge, in August 1923, only two damaging floods have occurred. The flood of March 1936 had two peaks

6 days apart. The first occurred on 12 March and amounted to 1340 c.f.s., and the second occurred on 18 March and amounted to 2520 c.f.s., which is the largest recorded rate of discharge. The flood of 21 September 1938 had a peak discharge of 1300 c.f.s. Lower peak rates of discharge have been recorded at this gaging station as follows: 10 January 1935, 1020 c.f.s.; 10 June 1931, 935 c.f.s.; 24 July 1938, 875 c.f.s.; 4 November 1927, 790 c.f.s.; and 7 April 1924, 740 c.f.s.

The volume of runoff during the second rise of March 1936 amounted to 5.81 inches and resulted from 6.1 inches of rainfall on the drainage area plus considerable snow melt. The flood of September 1938 had a volume of 4.26 inches and resulted from 8.9 inches of rainfall on the drainage area.

(3) <u>Frequency and Magnitude of Floods</u>. - To give some indication of the frequency, size, and seasonal occurrence of floods at Worcester, the stream flow record is analyzed in the following table.

TABLE 5. FLOOD OCCURRENCES - BLACKSTONE RIVER AT WORCESTER, MASS.

(Drainage area 31.3 square miles. Record Aug. 1923 - Sept. 1944)

Month	Peak rates of discharge, c.f.s. 700 to 1000	1000 to 1500	1500 to 2000	over 2500	Peak rate of discharge, c.f.s.
	Number of times equaled or exceeded				
January	2	1	0	0	1020
February	0	0	0	0	under 700
March	2	2	1	1	2520
April	1	0	0	0	740
May	0	0	0	0	under 700
June	1	0	0	0	935
July	1	0	0	0	875
August	0	0	0	0	under 700
Sept.	1	1	0	0	1300
October	0	0	0	0	under 700
November	2	0	0	0	790
December	0	0	0	0	under 700
Totals	10	4	1	1	2520

Frequency curves for the peak rates of discharge and the maximum daily rates appear on Plate No. I-4.

(4) <u>Watershed Characteristics</u>. - The total drainage area of the Blackstone River tributary to Leesville Pond is 31.3 square miles. Elevations vary from 485 feet above mean sea level at the pond to over 1300 feet at a few points on the rim of the basin. Above the pond the river breaks into two principal tributaries, Kettle Brook and Ramshorn Brook.

The first has its source in Kettle Brook Reservoir No. 4 of the Worcester Water Supply System, in the Town of Paxton, Mass., and flows in a general southeasterly direction about 8 miles to Leesville Pond. At its lower end where it flows through Stoneville Pond it is joined by Dark Brook coming in from the southwest. Ramshorn Brook flows from the south into the upper end of Leesville Pond.

The outstanding feature of the drainage area from a hydrological viewpoint is the large number of natural and artificial ponds and reservoirs and the large areas of flat swamp land. Thirteen reservoirs, five of which are operated for water supply purposes by the City of Worcester and eight of which are operated by private interests for industrial conservation storage, have a total capacity of 5.4 inches of runoff from the 31.3 square miles of drainage area tributary to the inlet of the proposed Worcester Diversion. The area and capacity of these reservoirs is shown in the following table:

TABLE 6. RESERVOIRS IN BLACKSTONE BASIN ABOVE

U.S.G.S. GAGING STATION

Name of pond	Use	Area Acres	Capacity, million cu. ft.
Kettle Brook Reservoir No. 4	W.S.	119	513
" " " No. 3	W.S.	37	152
" " " No. 2	W.S.	30	127
" " " No. 1	W.S.	15	19
Lynde Brook Reservoir	Cons.	132	715
Stoneville Reservoir	Cons.	88	185
Stoneville Pond	Cons.	45	100
Moss Reservoir (Eagle Pond)	Cons.	53	100
Dunns (Auburn) Pond	Cons.	26	40
Ramshorn Pond	Cons.	147	720
Ramshorn Meadow Pond	Cons.	38	22
Stone Crossing (Pondville) Pond	Cons.	45	125
Leesville (Trowbridgeville) Pond	Cons.	107	125
Total		882	2943

Surcharge storage in the ponds described above and storage in swamps act to produce floods having relatively low peaks and long "lag" times. This is confirmed by the unit hydrograph derived from the flood of September 1938, described later in this Appendix.

1-03. Design Flood for Worcester Diversion. - a. General. - The criterion adopted for the development of a design flood to fix the capacity of the Worcester Diversion tunnel was that it be a greater flood than any of record but of probable though very rare occurrence. The maximum flood of record for the 31.3 square mile drainage area tributary to the entrance of the proposed diversion tunnel is that of March 18, 1936, when the peak rate of discharge was 2520 c.f.s. According to the frequency magnitude curve shown on Plate No. I-4, this rate of discharge has an average recurrence interval of about 50 years. The second largest rate of discharge at the same point occurred on 12 March 1936, when a peak of 1340 c.f.s. was recorded. The third largest peak was 1300 c.f.s., which occurred on 21 September 1938. The first two peaks resulted from a combination of melting snow and rainfall and the third from rainfall alone. Because rainfall alone, during the summer and autumn months, is believed to produce a greater flood on an area of this size than melting snow and rain combined, the maximum rain storm of record in the vicinity, viz., that of 17-21 September 1938, was selected as a basis for the design flood. One of the two centers of maximum precipitation for this storm was in the vicinity of Barre, Mass., only 20 miles northwest of the center of the drainage area under study. The total depth of rain that fell on the 31.3 square miles of drainage area above Worcester from 17-21 September 1938 was computed at 8.92 inches, whereas the depth that fell on a similar area around Barre was computed at 15.55 inches. Since the average elevation of the Barre area is only about 200 feet higher than the average for the drainage area under study, it was considered reasonable to assume that with slightly different meteorological conditions the precipitation which occurred around Barre could have occurred over the drainage area above Worcester. Accordingly the actual record of precipitation, as it occurred on 31.3 square miles around Barre from 17 to 21 September 1938, was transposed to the 31.3 square-mile drainage area of the Blackstone River above the Leesville Dam and used as the design storm for computation of the design flood.

To compute the design flood from the transposed storm described above, a 3-hour unit hydrograph was derived from the actual storm and flood of record and applied to the rainfall excess of the transposed storm. Infiltration for the transposed storm was allowed at the same rates as actually computed in derivation of the unit hydrograph, and base flow was allowed on the same basis. The resulting design flood has a peak of 5600 c.f.s. at the 69th hour after the beginning of rainfall excess. Details of its computation follow.

b. Design Storm. - The first step in computation of the design storm was the development of an area-depth curve for the Barre, Mass., precipitation center. This was done by planimetering the isohyetals on a photostatically enlarged copy of Plate 1, U.S.G.S. Water Supply Paper No. 867. The resulting areas plotted against corresponding depths, together with the maximum recorded depth at the Barre rainfall station, gave the area-depth curve shown on Plate No. I-5. By planimetering the area under this curve the mean depth of rainfall on 31.3 square miles was determined to be 15.55 inches. By means of a mass rainfall curve, based upon the records of the Barre non-recording station and the Amherst, Mass. and Worcester, Mass. (Weather Bureau) recording stations a storm pattern for the 15.55 inches of rainfall was developed as shown in columns (2) and (3) of Table 7, which follows. Infiltration was assumed at approximately the same rates that were determined to have occurred during this storm on the Blackstone drainage area, viz., 0.16 inch per hour for the first 49 hours then decreasing uniformly to a minimum value of 0.05 inch per hour at the end of the storm. This infiltration pattern grouped in 3-hour periods is shown in column (4). The resulting rainfall-excess pattern used to compute the design flood is shown in column (5).

TABLE 7. DESIGN STORM - WORCESTER DIVERSION

Time in Hours	Accumulated depth of rainfall, inches	Inches depth each 3 hours		
		Rainfall	Infiltration	Rainfall excess
3	.04	.04	.04	-
6	.08	.04	.04	-
9	.17	.09	.09	-
12	.30	.13	.13	-
15	.70	.40	.35	.05
18	1.00	.30	.30	-
21	1.08	.08	.08	-
24	1.10	.02	.02	-
27	1.40	.30	.23	.07
30	1.78	.38	.36	.02
33	2.10	.32	.32	-
36	2.19	.09	.09	-
39	2.48	.29	.21	.08
42	2.61	.13	.13	-
45	2.67	.06	.06	-
48	5.02	2.35	.48	1.87
51	5.98	.96	.46	.50
54	6.18	.20	.20	-
57	6.30	.12	.12	-
60	7.36	1.06	.26	.80
63	9.70	2.34	.33	2.01
66	12.90	3.20	.29	2.91
69	13.96	1.06	.26	.80
72	14.33	.37	.13	.24
75	14.41	.08	.07	.01
78	15.47	1.06	.16	.90
81	15.53	.06	.05	.01
84	15.55	.02	.02	-
Totals	-	15.55	5.28	10.27

c. Unit Hydrograph. - A 6-hour unit hydrograph, for the 31.3 square mile drainage area under study, was derived from the storm and flood records of September 1938. Details of this derivation are shown on Plates No. I-6, No. I-7, and No. I-8. The resulting 3-hour unit hydrograph, determined from the S-curve of the 6-hour unit hydrograph, has a peak rate of discharge at the 18th hour of 990 c.f.s. or 31.6 c.f.s. per square mile, and is shown on Plate No. I-9. The Snyder $LL_{ca})^{0.3}$ value for the drainage area is 3.65, which in conjunction with the 16.5-hour "lag" gives a Snyder C_t of 4.5. This is high, but a high value is to be expected in view of the large amount of storage in the drainage area. The corresponding Snyder C_p x 640 value is 521, which is only a little above Snyder's average values. The discharge rates for the derived 3-hour unit hydrograph are shown in Table 8 for each 3-hour interval from the beginning of rainfall excess.

TABLE 8. RATES OF DISCHARGE FOR 3-HOUR UNIT HYDROGRAPH DERIVED FROM SEPTEMBER 1938 FLOOD.

Blackstone River at Worcester, Mass. - 31.3 square miles

Time in hours	Discharge in c.f.s.	Time in hours	Discharge in c.f.s.	Time in hours	Discharge in c.f.s.
0	0	51	186	102	51
3	1	54	170	105	47
6	2	57	155	108	43
9	5	60	141	111	40
12	12	63	128	114	36
15	260	66	116	117	33
18	990	69	105	120	30
21	682	72	98	123	27
24	490	75	90	126	25
27	392	78	85	129	22
30	336	81	79	132	19
33	300	84	75	135	16
36	275	87	71	138	13
39	256	90	66	141	10
42	238	93	63	144	4
45	219	96	59	147	0
48	203	99	55		

d. Design Flood Hydrograph. - The hydrograph of the design flood, as it will flow into Leesville Pond, was computed by applying the 3-hour unit hydrograph shown in Table 8 to the 3-hour rainfall-excess depths shown in column (5) of Table 7 and adding the base flow of the stream as it occurred during the September 1938 flood. Leesville Pond is assumed to be filled to the spillway crest and no discharge over the Leesville Spillway is planned. Hence no reservoir or surcharge storage will be available in Leesville Pond, and the Worcester Diversion tunnel and channel must be adequate to pass the maximum rate of inflow without having the pool elevation in Leesville Pond rise above the spillway crest. Computed rates of discharge for the design flood are shown in Table 9 and the hydrograph of this flood is shown on Plate No. I-10.

TABLE 9. DESIGN FLOOD, WORCESTER DIVERSION - COMPUTED

RATES OF DISCHARGE

Time in hours	Discharge in c.f.s.	Time in hours	Discharge in c.f.s.	Time in hours	Discharge in c.f.s.
0	33	81	3730	162	485
3	34	84	3180	165	450
6	35	87	2800	168	420
9	37	90	2530	171	385
12	39	93	2320	174	360
15	53	96	2140	177	325
18	91	99	1980	180	295
21	77	102	1820	183	270
24	70	105	1680	186	245
27	84	108	1550	189	220
30	140	111	1430	192	190
33	130	114	1310	195	160
36	115	117	1210	198	140
39	130	120	1130	201	135
42	185	123	1050	204	130
45	175	126	980	207	125
48	625	129	925	210	120
51	2110	132	875		
54	1900	135	825		
57	1400	138	785		
60	1340	141	745		
63	2290	144	705		
66	4140	147	665		
69	5630	150	630		
72	4870	153	590		
75	3960	156	550		
78	3490	159	520		

e. Tailwater Conditions in Blackstone River. - The Worcester Diversion will consist of a tunnel from Leesville Pond to the head of Hull Brook and an open channel from the tunnel exit to the Blackstone River. The proposed location for the lower end of this channel is at river mile 46.38 on the Blackstone River, approximately 1200 feet upstream from the Greenwood Street Bridge. A rating curve for the Blackstone River at mile 46.38 has been computed from flood stages of record and estimated corresponding rates of discharge. It is shown on Plate No. I-11.

The total drainage area tributary to mile 46.38 is 69.8 square miles, and the net drainage area below the Worcester Diversion is 38.5 square miles. When the Worcester Diversion is operating at maximum capacity, the stage of the Blackstone River at mile 46.28 will be governed by the combined discharge of the Worcester Diversion and the net drainage area of 38.5 square miles. Accordingly, the maximum discharge from the net drainage area under design storm conditions and the resulting maximum tailwater elevation were estimated as follows:

From the area-depth curve of the transposed design storm shown on Plate I-5, the average rainfall on the 38.5 square miles constituting the net drainage area was computed at 14.71 inches. Using the characteristics of the 3-hour unit hydrograph derived for the 31.3 square mile area, a similar 3-hour unit hydrograph was derived for the 38.5 square mile net area, having a peak of 950 c.f.s. at the 21st hour. A storm pattern for the 14.71 inches of rainfall was developed in a manner similar to that used for the design flood, and the 3-hour unit hydrograph was then applied to the rainfall excess, which amounted to 9.60 inches. A base flow proportionate to that used for the inlet design flood was added. The resulting hydrograph of runoff from the net drainage area of 38.5 square miles had a peak rate of discharge of 5800 c.f.s. occurring at the same time as the maximum discharge of the diversion works. Hence the maximum discharge in the Blackstone River just below the terminus of the diversion channel is estimated to be 11,400 c.f.s. when the diversion channel is discharging 5600 c.f.s. From the rating curve referred to above, the river stage for this rate of discharge would be elevation 417.5 m.s.l. at river mile 46.38.

1-04. Computed Spillway Flood for Leesville Dam. - a. General A computed spillway flood was developed for the Leesville Dam for the purpose of examining the safety of the dam under the condition of maximum possible discharge. The maximum possible precipitation on 31.3 square miles in the adjoining Thames River Basin was adopted for computation of the forecast worst storm. The unit hydrograph developed from the September 1938 flood was not considered adequate to represent runoff conditions during a flood of spillway design magnitude. Accordingly, a synthetic unit hydrograph was developed having a shorter lag time and higher peak rate of discharge. Infiltration was allowed at the 0.05 inch per hour rate customarily used for computation of spillway design floods in this region.

b. Forecast Worst Storm. - The maximum possible rainfall for a drainage area of 31.3 square miles in the adjoining Thames River Basin, as computed by the Hydrometeorological Section, is 23.09 inches in 24 hours. Its distribution with respect to time is shown in column (2) of Table 10, as taken from the depth-area duration curves for the Thames River Basin prepared by the Hydrometeorological Section. Column (4) of the same table shows the depth of rainfall for each period listed in column (3). The computed infiltration for each period, at 0.05 inch per hour, is given in column (5) and the resulting rainfall excess is shown in column (6). In columns (7) and (8) the same rainfall excess is arranged according to the most severe storm pattern, which was used to obtain the computed spillway flood.

TABLE 10. MAXIMUM POSSIBLE PRECIPITATION AND FORECAST WORST STORM

Blackstone River at Worcester, Mass. - 31.3 square miles

Duration of rainfall in hours	Depth of rainfall in inches	Periods in hours	Depth of rainfall each period inches	Infiltration each period inches	Rainfall excess each period inches	Storm pattern: Length of period: hours	Storm pattern: Rainfall excess each period, inches
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	4.66	1	4.66	.05	4.61		
3	11.63	2	6.97	.10	6.87	3	0.09
6	19.32	3	7.69	.15	7.54	3	0.55
9	21.57	3	2.25	.15	2.10	3	7.54
12	22.27	3	0.70	.15	0.55	1	4.61
15	22.62	3	0.35	.15	0.20	2	6.87
18	22.86	3	0.24	.15	0.09	3	2.10
21	23.00	3	0.14	.14	-	3	0.20
24	23.09	3	0.09	.09	-		
Totals			23.09	1.13	21.96		21.96

c. Unit Hydrograph. - A 3-hour unit hydrograph having a peak rate of discharge of 1230 c.f.s. and a lag of 13 hours was developed synthetically for use in obtaining the computed spillway flood by means of the forecast worst storm. This peak rate of discharge is 25% greater and the lag approximately 20% less than for the 3-hour unit hydrograph derived from the flood of September 1938. The more severe runoff conditions represented in the synthetic unit hydrograph reflect the fact that surcharge storage in the reservoirs, ponds and swamps will have relatively less modifying effect during an extremely large flood than during a considerably smaller flood such as that of September 1938. For the smaller flood the spillways provided will hold water in surcharge storage without overtopping the dams, but for a flood as large as the computed spillway flood the dams would be overtopped, and a shortening of the lag time and increase in rates of discharge would result.

The peak rate of discharge for the derived unit hydrograph (990 c.f.s.) was increased 25% to obtain the peak of the synthetic unit hydrograph. The lag for the synthetic unit hydrograph was fixed at 13 hours so as to keep the $C_p \times 640$ value at 521, which is the same as for the derived unit hydrograph. The resulting 3-hour unit hydrograph is shown on Plate No. I-9, and the discharge rates are given in the following table.

TABLE 11. RATES OF DISCHARGE FOR SYNTHETIC 3-HOUR UNIT HYDROGRAPH USED TO OBTAIN COMPUTED SPILLWAY FLOOD

Blackstone River at Worcester, Mass. - 31.3 square miles

Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.	Time, hours	Discharge c.f.s.
0	0	24	394	72	69
1	4	25	370	75	64
2	12	26	349	78	58
3	25	27	331	81	53
4	41	28	314	84	49
5	62	29	299	87	44
6	89	30	285	90	40
7	124	31	272	93	35
8	170	32	260	96	31
9	229	33	248	99	28
10	303	34	237	102	24
11	393	35	226	105	21
12	578	36	216	108	18
13	941	39	192	111	15
14	1186	42	173	114	12
15	1221	45	155	117	11
16	988	48	142	120	9
17	795	51	130	123	7
18	665	54	119	126	6
19	581	57	107	129	4
20	526	60	98	132	3
21	485	63	90	135	1
22	451	66	83	138	0
23	421	69	75		

d. Computed Spillway Flood Hydrograph. - The inflow hydrograph of the computed spillway flood was obtained by applying the 3-hour unit hydrograph shown in Table 11 to the rainfall excess for the forecast worst storm, shown in column (8) of Table 10, and adding a base flow of 60 c.f.s. The resulting hydrograph has a peak of 20,900 c.f.s. at the 23d hour. It is shown graphically on Plate No. I-12, and the discharge rates are given in Table 12 which follows:

STATION	OPERATED BY	RECORD BEGINS
Barre	Mass. Dept of Public Health	1919
Boylston	Met. Dist Water Supply Comm.	1896
Charlton Depot	Mass. Dept of Public Health	1920
Clinton	Met. Dist. Water Supply Comm.	1902
Cold Brook	Met. Dist. Water Supply Comm.	1919
Holden No. 2	City of Worcester	1885
Hubbardston	Mass. Dept. of Public Health	1919
Jefferson	Met. Dist. Water Supply Comm.	1897
Kendall Reservoir	City of Worcester	1913
Kettle Brook No. 3	City of Worcester	1905
Lynde Brook	City of Worcester	1875
Millbury	New England Power Co.	1930
Northbridge	Mass. Dept. of Public Health	1872
North Rutland	Mass. Dept. of Public Health	1924
Princeton	Met. Dist. Water Supply Comm.	1884
Rutland	Mass. Sanatorium	1902
Southbridge	Southbridge Water Supply Co.	1912
Sterling	Met. Dist. Water Supply Comm.	1897
Uxbridge	Mass. Dept. of Public Health	1931
Webster	Mass. Dept. of Public Health	1880
West Rutland	Met. Dist. Water Supply Comm.	1928
Williamsville	Mass. Dept of Public Health	1924
Worcester	Winter Hill Meteorological Observatory	1841
Worcester No. 2	City of Worcester	1895

BLACKSTONE RIVER FLOOD CONTROL

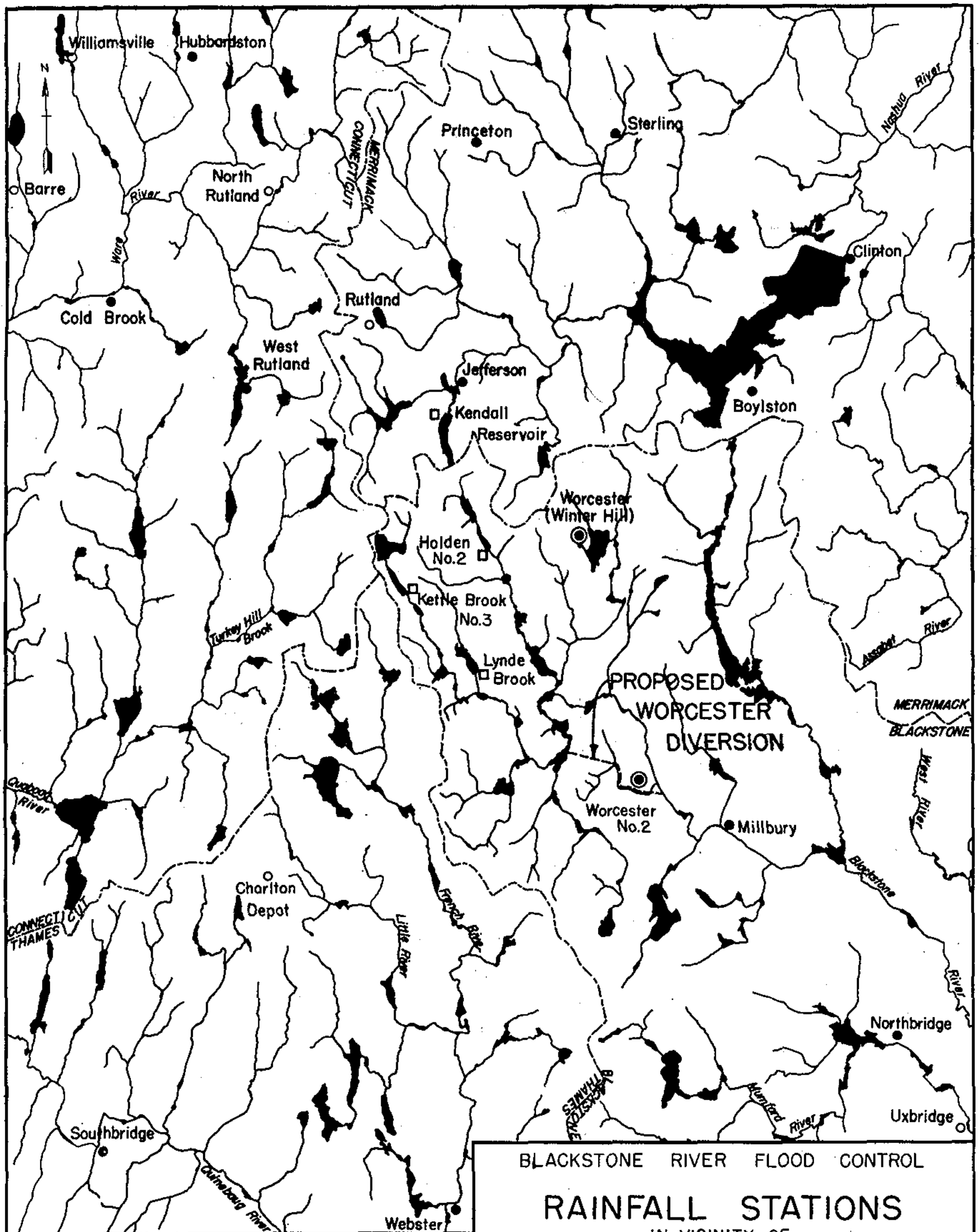
RAINFALL STATIONS

IN VICINITY OF

WORCESTER, MASSACHUSETTS

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

MARCH 1946



BLACKSTONE RIVER FLOOD CONTROL

RAINFALL STATIONS
IN VICINITY OF
WORCESTER, MASSACHUSETTS

SCALE OF MILES

2 1 0 2 4 6 8

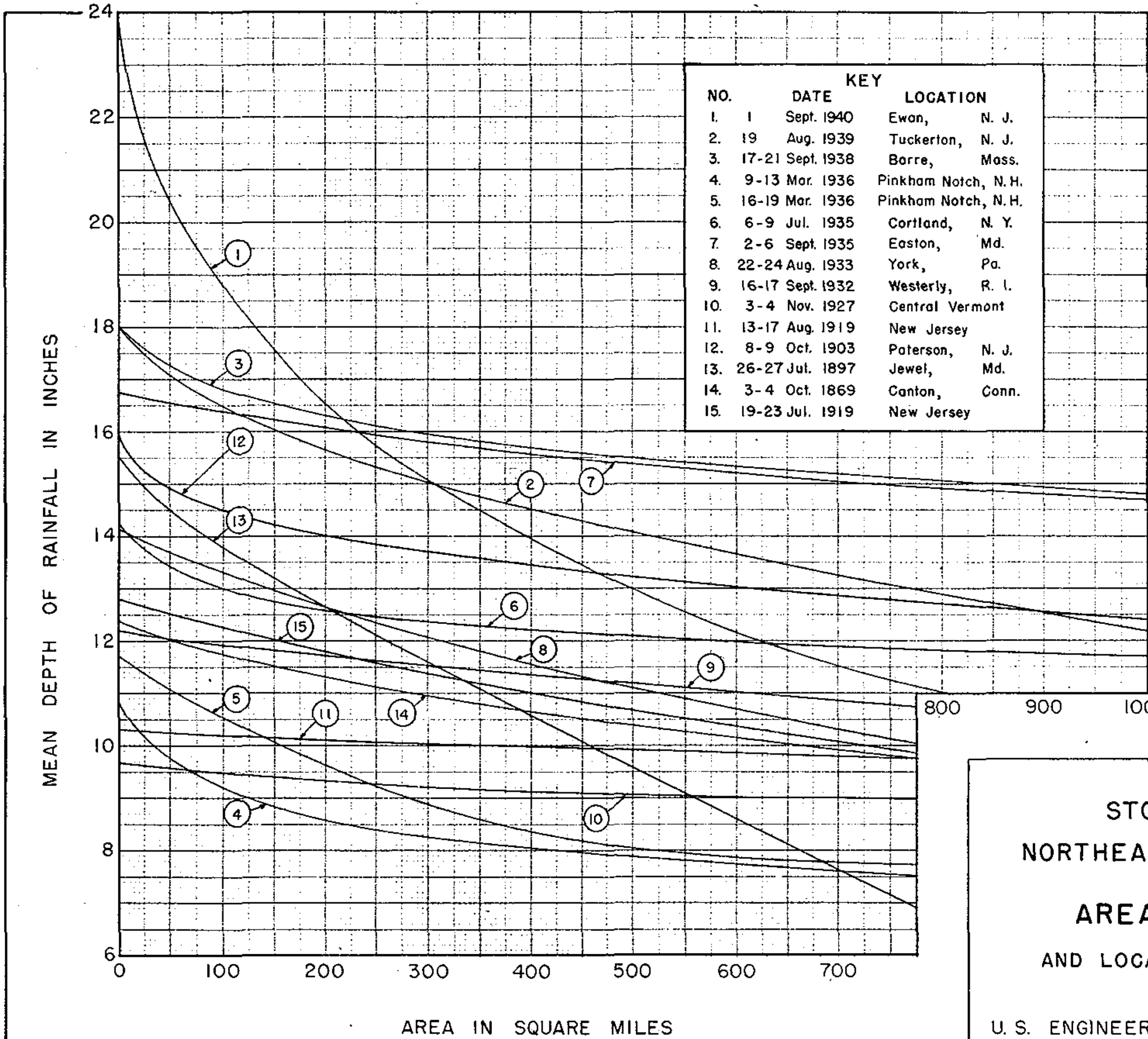
U.S. ENGINEER OFFICE PROVIDENCE, R.I.

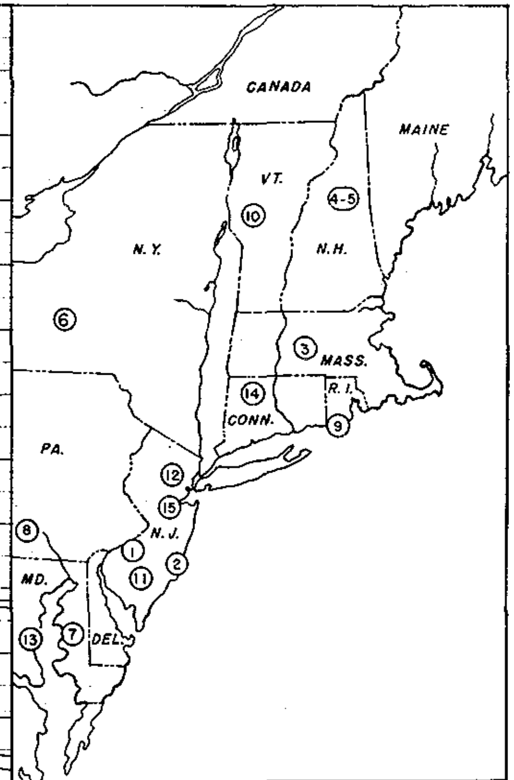
MARCH 1946

LEGEND

Records published by U.S. Weather Bureau:
- ◉ Recording stations
- ● Non-recording stations
- ○ Records available from Mass. Dept. of Public Health
- □ Records available from City of Worcester
- —·—·— Watershed Boundaries

APPENDIX I PLATE NO. 3





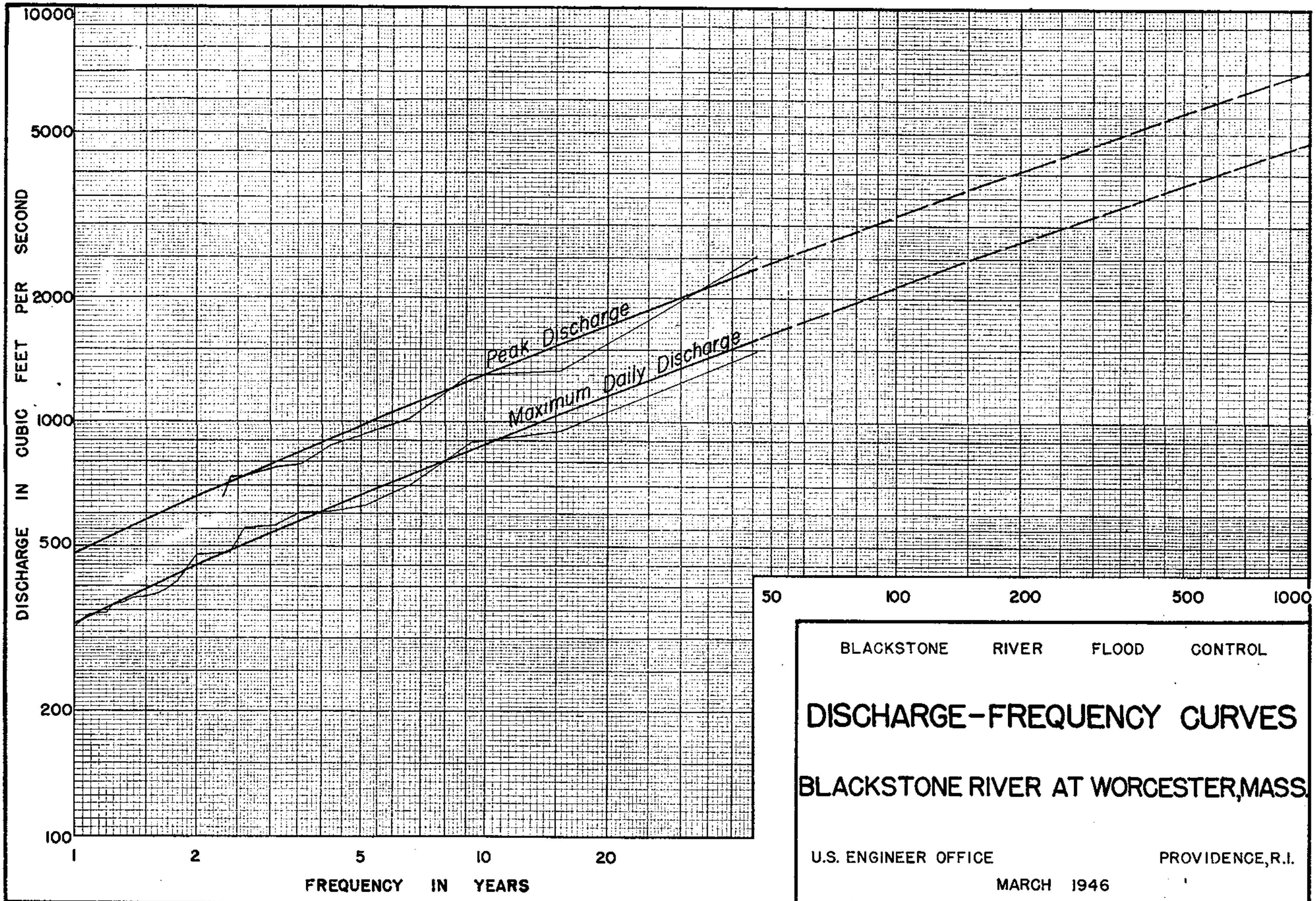
LOCATION OF STORM CENTERS

STORMS OF RECORD
NORTHEASTERN UNITED STATES

AREA - DEPTH CURVES

AND LOCATION OF STORM CENTERS

U. S. ENGINEER OFFICE PROVIDENCE, R. I.



BLACKSTONE RIVER FLOOD CONTROL

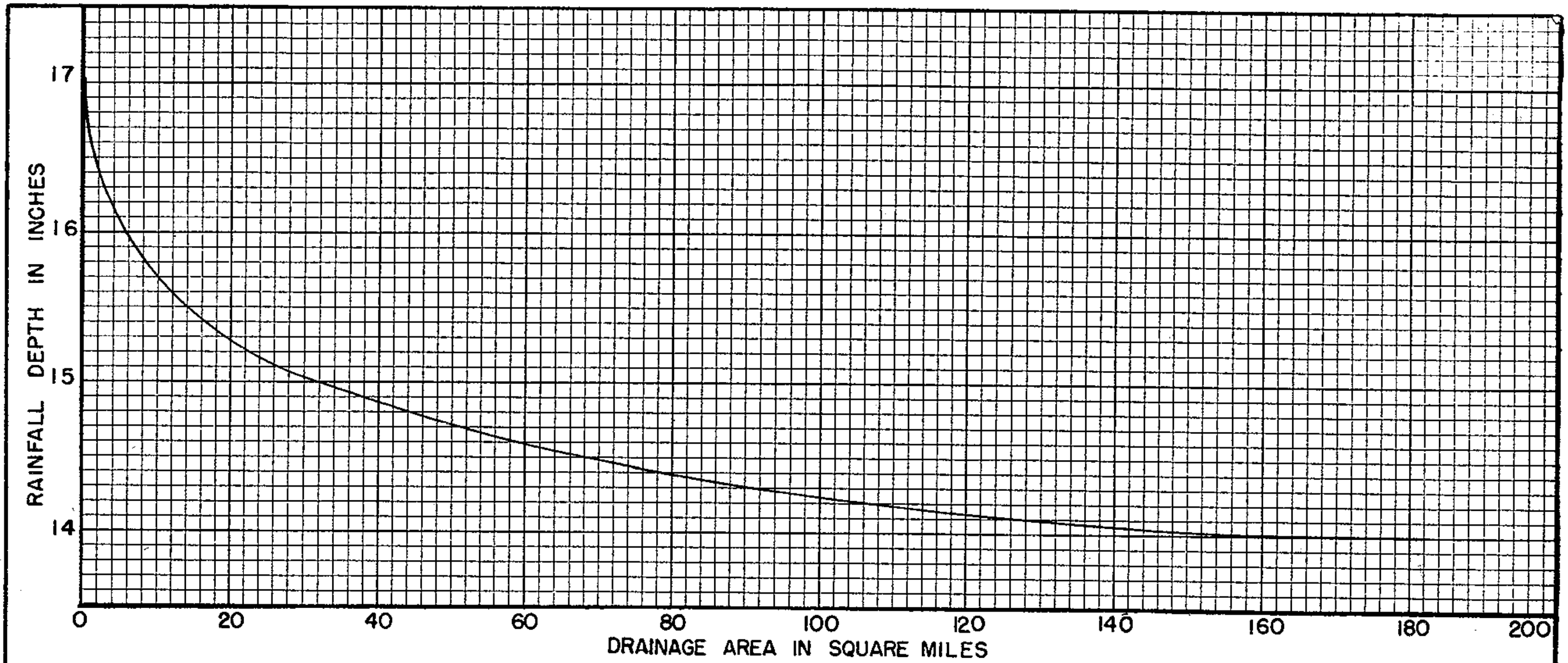
DISCHARGE-FREQUENCY CURVES

BLACKSTONE RIVER AT WORCESTER, MASS.

U.S. ENGINEER OFFICE — PROVIDENCE, R.I.

MARCH 1946

WORCESTER DIV. DPR



BLACKSTONE RIVER FLOOD CONTROL

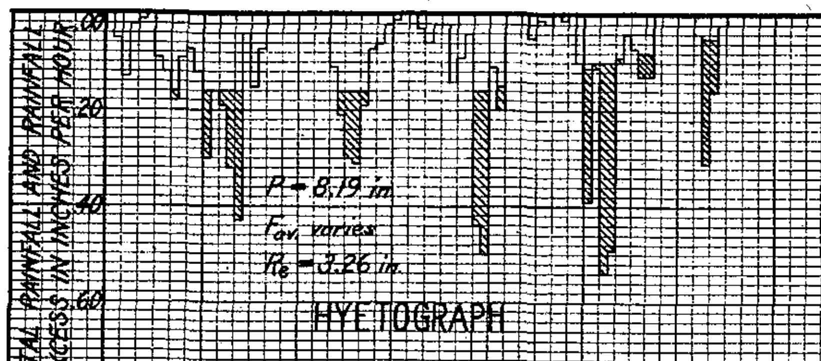
AREA-DEPTH CURVE

SEPTEMBER 1938 STORM CENTER NEAR BARRE, MASSACHUSETTS

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

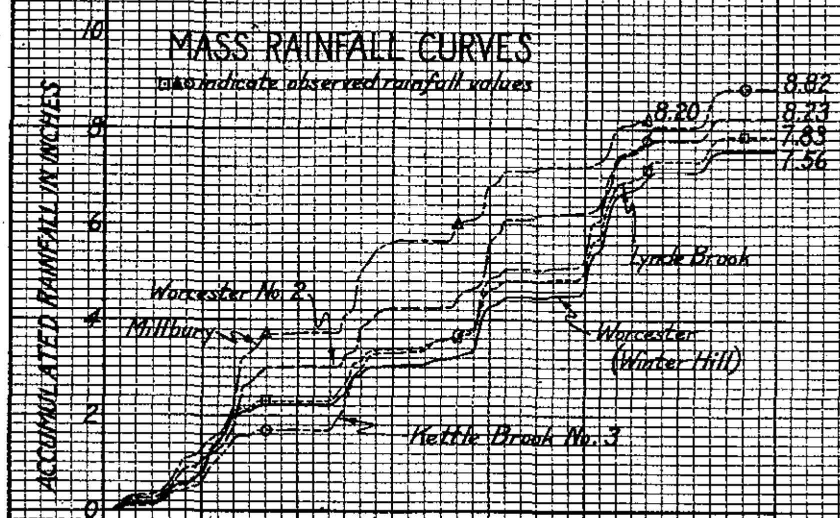
MARCH 1946

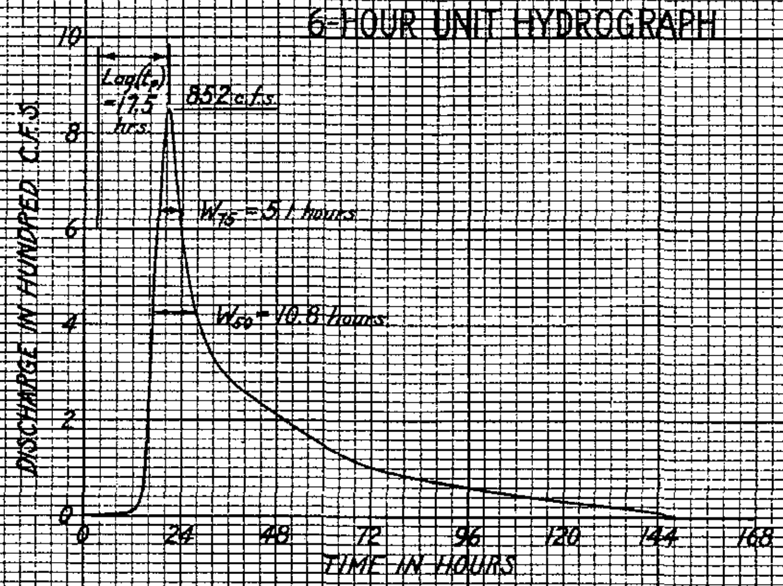
APPENDIX I PLATE NO. 5



SUMMARY OF PRECIPITATION DATA
By 6-hour periods

Period ending	18 8PM	19 2AM	19 8AM	19 2PM	19 8PM	20 2AM	20 8AM	20 2PM	20 8PM	21 2AM	21 8AM	21 2PM	21 8PM	Total
Rainfall	.42	.92	1.20	.90	.93	.35	.27	1.38	.08	.61	1.37	.13	.53	8.19
Infiltration	.42	.76	.74	.00	.59	.32	.27	.71	.08	.32	.48	.08	.16	4.93
Rainfall excess	.00	.16	.46	.00	.34	.03	.00	.67	.00	.29	.89	.05	.37	3.26





NOTES

Drainage area = 31.3 square miles

Weight by Thiessen areas:

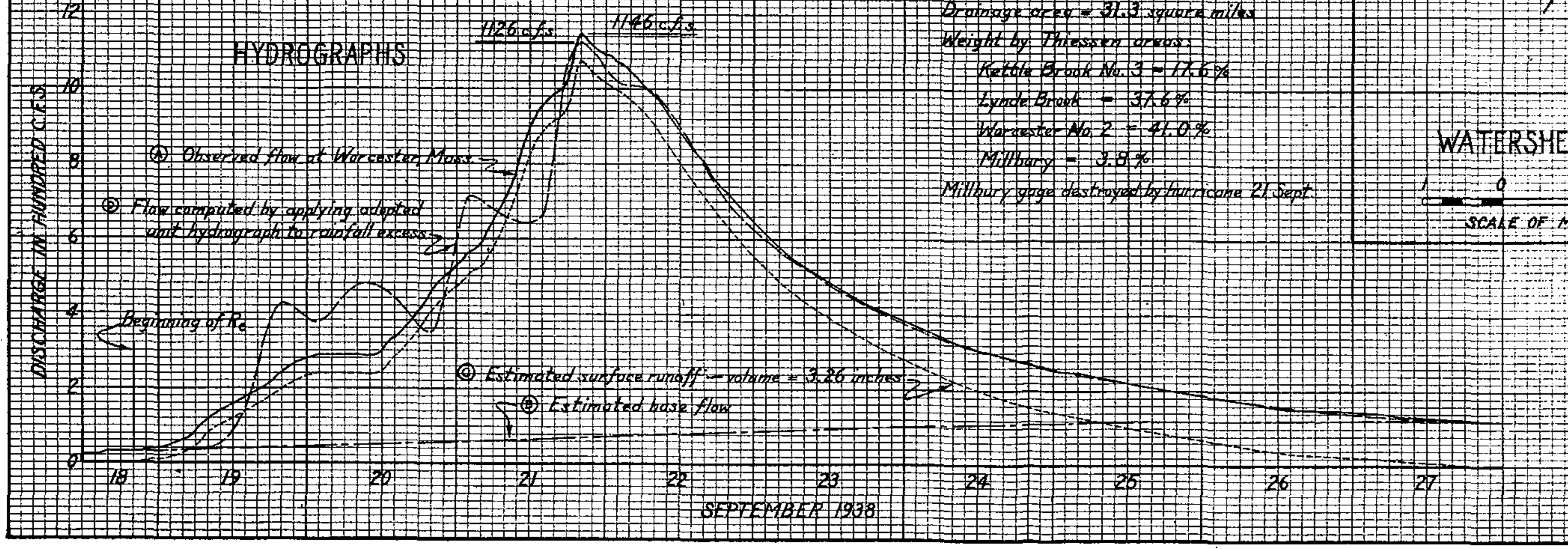
Kettle Brook No. 3 = 17.6%

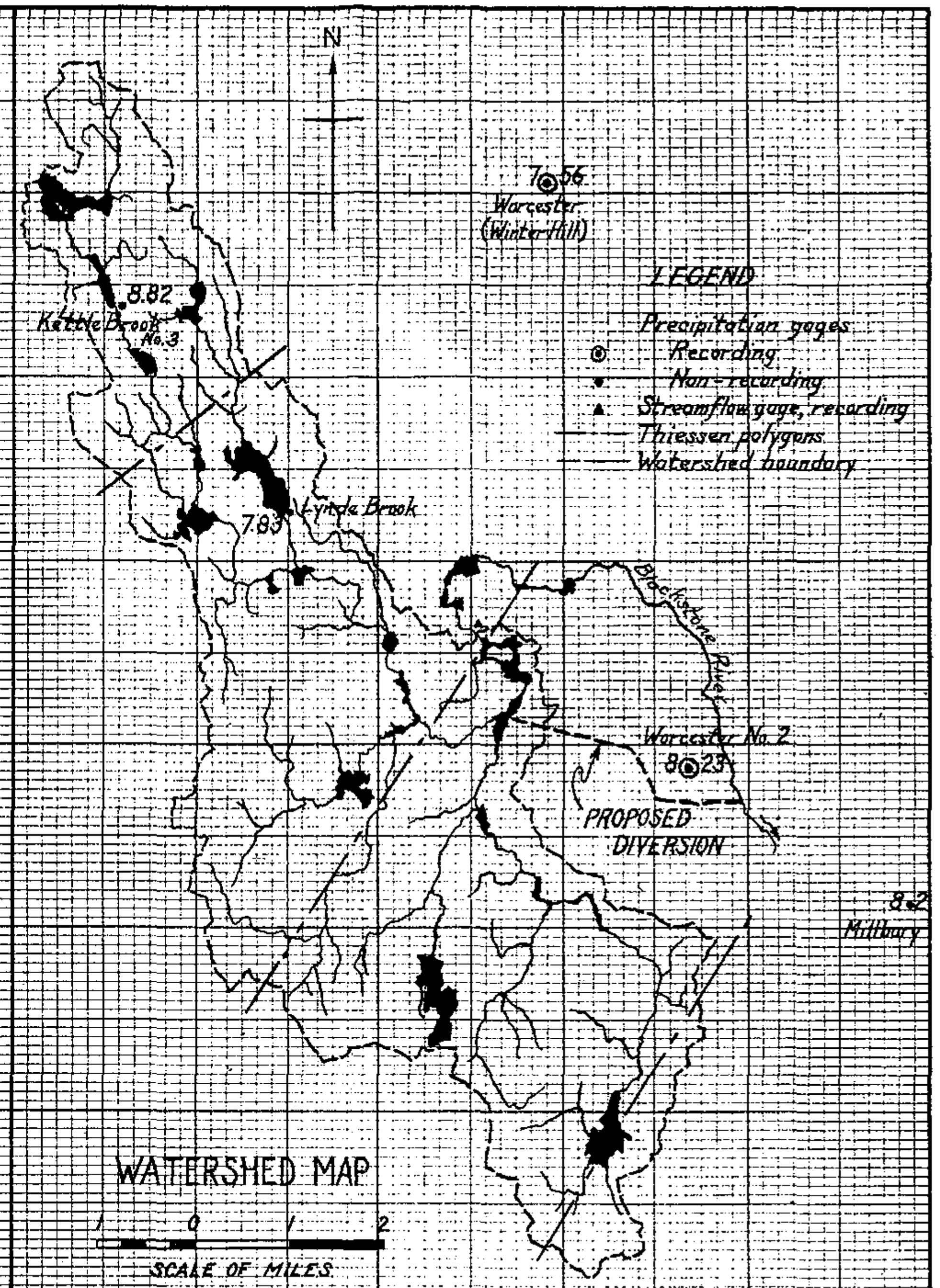
Lynde Brook = 37.6%

Worcester No. 2 = 41.0%

Millbury = 3.8%

Millbury gage destroyed by hurricane 21 Sept.





BLACKSTONE RIVER AT WORCESTER, MASSACHUSETTS
(AT UPSTREAM END OF WORCESTER DIVERSION)

UNIT HYDROGRAPH DETERMINATION

STORM OF 18-21 SEPTEMBER 1938

IN 3 SHEETS — SHEET NO. 1

SCALES: AS SHOWN

U.S. ENGINEER OFFICE — PROVIDENCE, R.I.

MARCH 1946 — DRAWN: R.F. — CHECKED: — FILE NO. BE-3-

WAR DEPARTMENT — CORPS OF ENGINEERS, U. S. ARMY

Computed by RF
Date Feb. 1946

UNIT HYDROGRAPH DETERMINATION

SHEET NO. 2

STREAM Blackstone River LOCATION Worcester, Mass.

DRAINAGE AREA 31.3 SQ. MI.

STORM OF 18-21 Sept. 1938 PREPARED BY Providence DIST. N.E. DIV.

AV. RAINFALL, 8.19 INCHES; RAINFALL-EXCESS, 3.26 INCHES; F_{av}, VARIABLE IN./HR.

* L 13.4 mi.; L_{ca} 5.6 mi.; $(LL_{ca})^{0.3}$ 3.65; t_R 6 hrs.;
LAG (t_p) 17.5 hrs.; C_t 4.79; q_p 27.2 c. f. s./sq.mi.; C_p640 476

LAG (t_{cmR}) ______ hrs.; W_{50} 10.8 hrs.; W_{75} 5.1 hrs.; SLOPE ______

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	DERIVED 6-HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED 6-HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
18-8P	32	32	0	0	0	32
					0	
19-2A	50	34	16	2	1	34
					3	
8	120	37	83	8	9	39
					136	
19-2P	179	40	139	625	625	144
					836	
8	241	43	198	586	586	426
					441	
20-2A	287	46	241	364	364	377
					318	
8	290	49	241	284	288	474
					266	
20-2P	336	52	284	248	247	442
					229	
8	462	56	406	213	211	351
					194	
21-2A	564	59	505	177	178	713
					162	
8	705	63	642	148	148	658
					134	
21-2P	953	67	886	122	122	666
					111	
8	1146	71	1075	102	102	1126
					94	
22-2A	1072	75	997	87	87	1019
					82	
8	980	80	900	77	77	986
					73	
22-2P	841	84	757	69	69	848
					64	
8	719	88	631	61	61	694
					57	
23-2A	614	92	522	53	53	597
					49	
8	530	95	435	45	45	523
					42	
23-2P	467	98	369	38	38	461
					34	
8	414	101	313	31	31	407

* FOR USAGE SEE "SYNTHETIC UNIT GRAPHS" - TRANS. A. G. U., PART I, 1938.

Computed by RF
Date Feb. 1946

UNIT HYDROGRAPH DETERMINATION

SHEET NO. 3

STREAM Blackstone River LOCATION Worcester, Mass.

DRAINAGE AREA 31.3 SQ. MI.

STORM OF 18-21 Sept. 1938 PREPARED BY Providence DIST. N.E. DIV.

AV. RAINFALL 8.19 INCHES; RAINFALL-EXCESS 3.26 INCHES; F_{av} VARIABLE IN./HR.

* { L 13.4 mi.; L_{ca} 5.6 mi.; $(LL_{ca})^{0.3}$ 3.65 ; t_R 6 hrs.;
LAG (t_p) 17.5 hrs.; C_t 4.79 ; q_p 27.2 c. f. s./sq.mi.; C_p 640 476 }

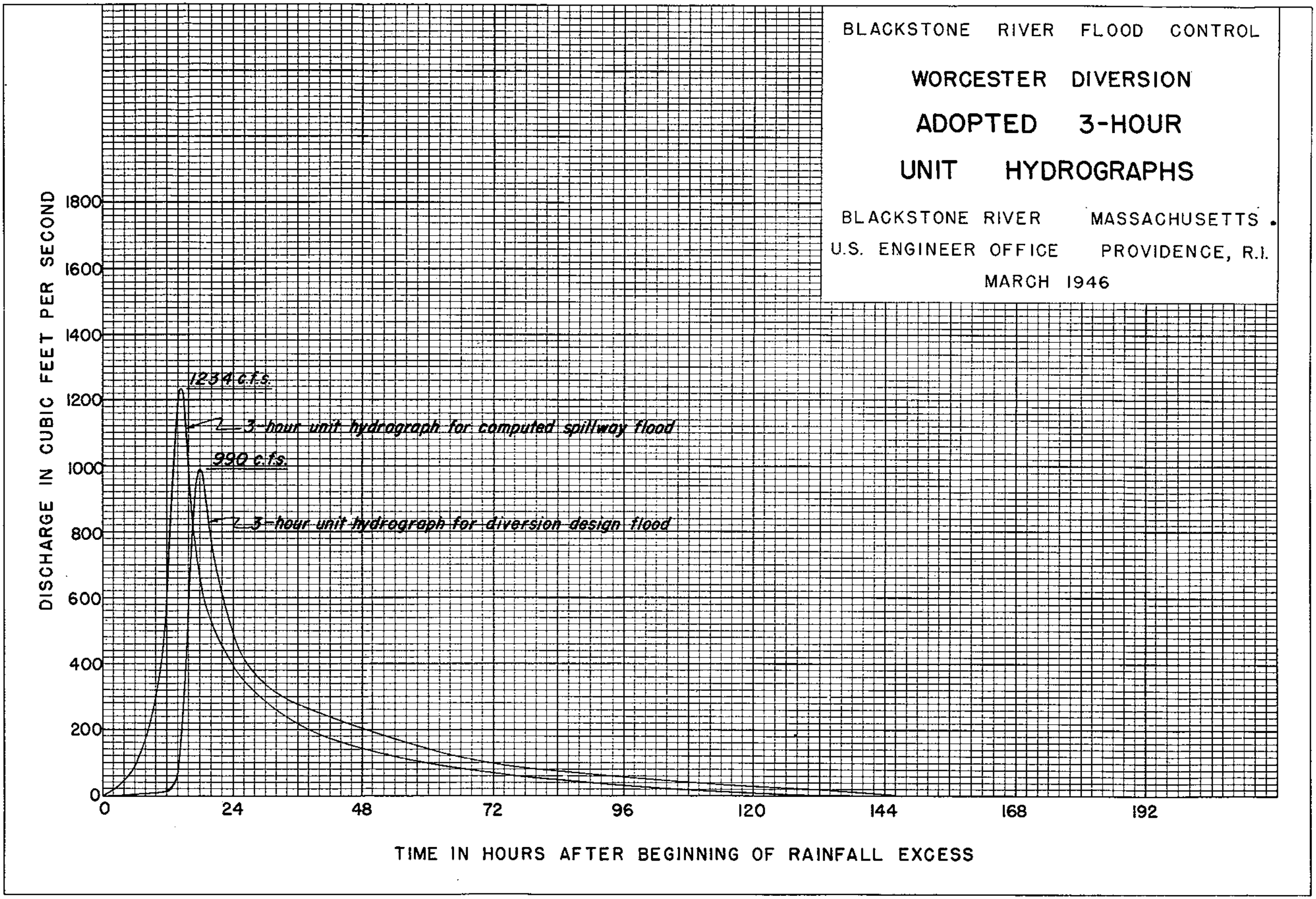
LAG (t_{cmR}) ______ hrs.; W_{50} 10.8 hrs.; W_{75} 5.1 hrs.; SLOPE ______

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	DERIVED 6-HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED 6-HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
24-2A	371	104	267	26	26	362
					23	
8	330	107	223	21	21	326
					18	
24-2P	300	110	190	15	15	301
					11	
8	271	113	158	8	7	276
					2	
25-2A	253	115	138	0	0	252
8	235	117	118			233
25-2P	215	118	97			215
8	200	119	81			198
26-2A	184	120	64			185
8	167	120	47			170
26-2P	154	121	33			156
8	149	121	28			147
27-2A	144	122	22			138
8	134	122	12			128
27-2P	127	122	5			125
8	122	122	0			122

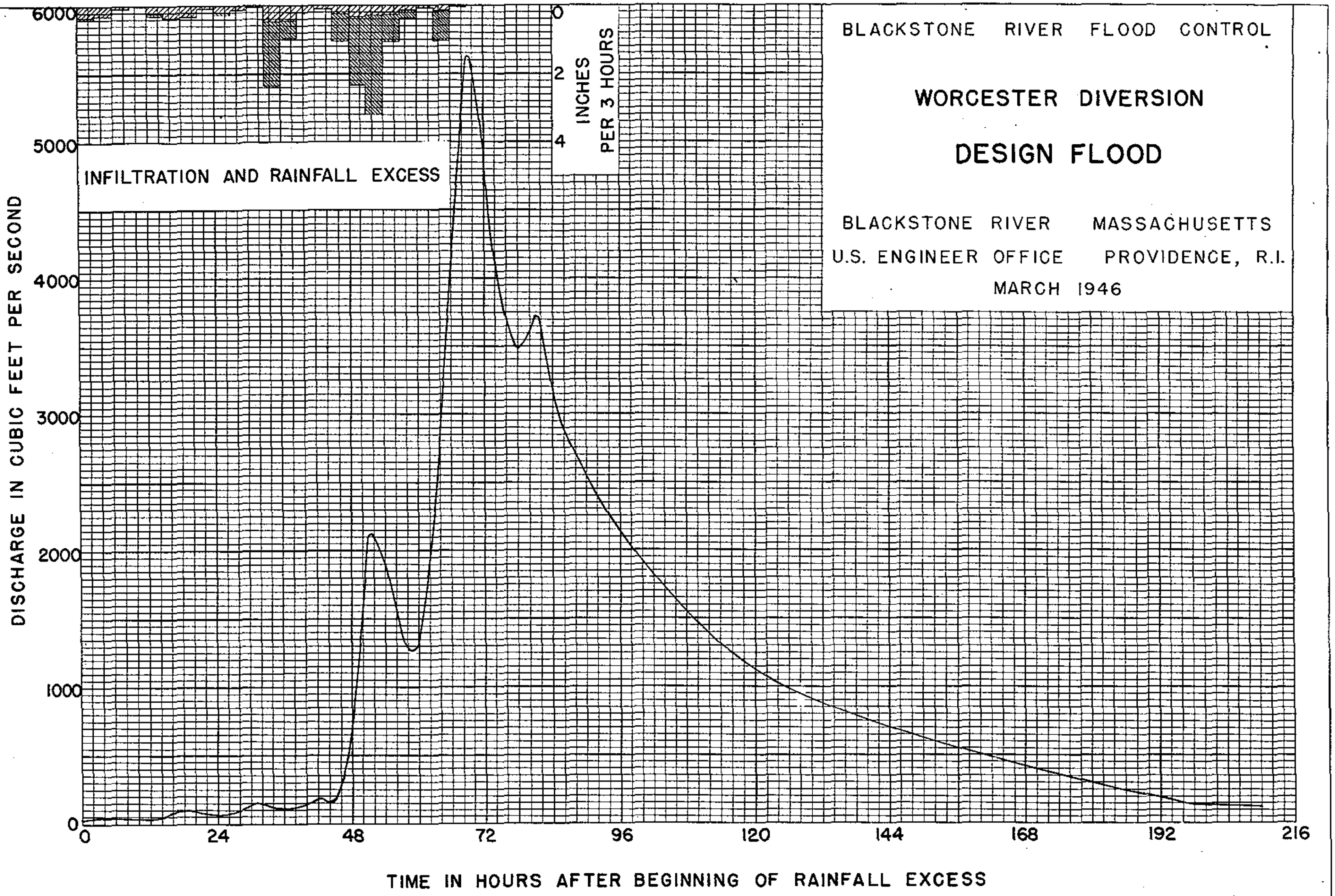
* FOR USAGE SEE "SYNTHETIC UNIT GRAPHS" - TRANS. A. G. U., PART I, 1938.

WORCESTER DIV. DPR

APPENDIX I PLATE NO.9



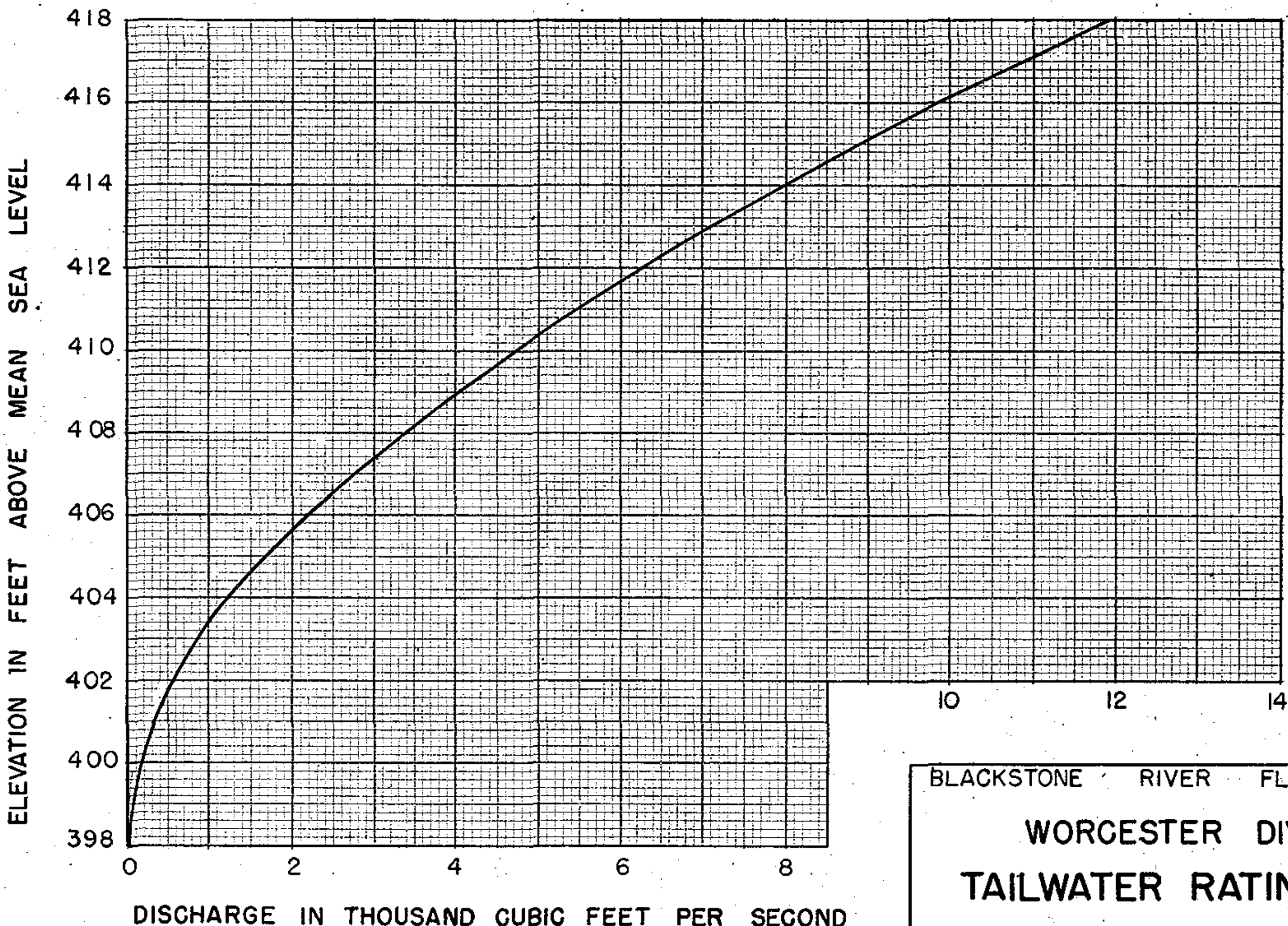
WORCESTER DIV. DPR



APPENDIX I PLATE NO.10

WORCESTER DIV. DPR

APPENDIX I PLATE NO. 11



NOTE: This curve applies at section 3000 feet downstream from bridge, U. S. Highway No. 20.

BLACKSTONE RIVER FLOOD CONTROL

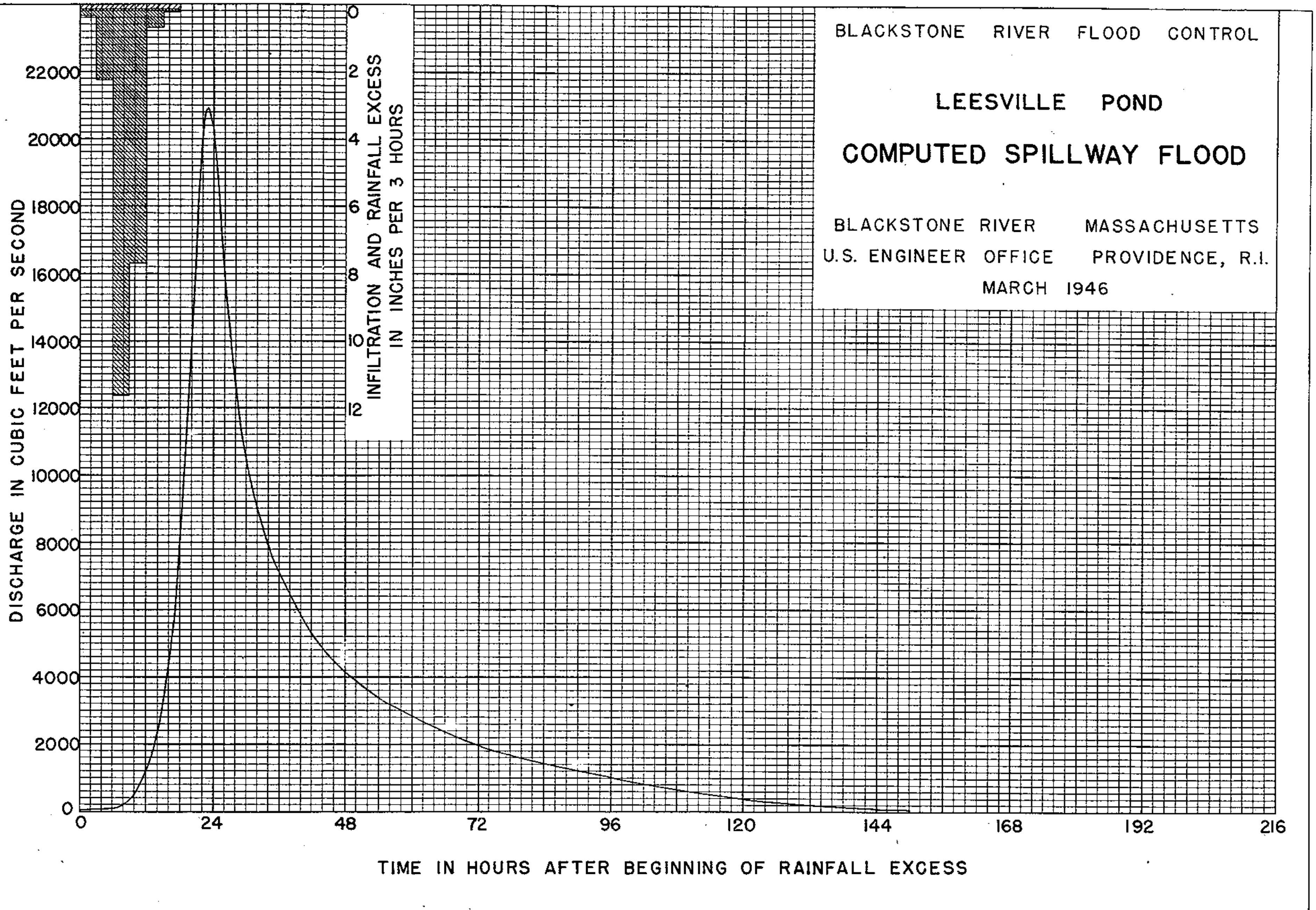
WORCESTER DIVERSION

TAILWATER RATING CURVE

BLACKSTONE RIVER, MASSACHUSETTS

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

MAY 1946



WORCESTER DIV. DPR

APPENDIX I PLATE NO. 12

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX II

GEOLOGY

To Accompany Definite Project Report

dated Sept. 1946

WORCESTER DIVERSION

APPENDIX II. GEOLOGY

CONTENTS

WORCESTER DIVERSION

TABLE

PLATES

WORCESTER DIVERSION

APPENDIX II. GEOLOGY

2-01. General Topography and Geology of Area. - Worcester Diversion, including drainage area, tunnel and return channel, is located in the northwest part of the Blackstone watershed and in the southeast quarter of Worcester County, Massachusetts. The area is located near the southeast edge of the central upland of Massachusetts, locally designated as the Worcester County plateau, whose bedrock consists of alternate bands of granite and narrower bands of folded metamorphosed sediments. Structurally, the project is within the synclinal Worcester trough. The dissected upland is a plateau only to the extent that the upland areas of comparatively smooth surfaces and the tops of the greater part of the interstream and interlowland areas are at an accordant elevation. The area has a maximum relief of approximately 300 feet, exclusive of Asnebumskit Hill, a monadnock in the north-central part of the drainage area. In the vicinity, valleys occur between 400 and 500 feet above sea level and ridges at about 700 feet above sea level. The trend of the ridges and valleys is structurally controlled by the bedrock formations, which have a general northeast-southwest grain. Locally, ridges are oriented in the direction of glacier advance.

a. The drainage area will be formed in the pre-glacial Auburn valley, a major regional pre-glacial drainage course. Within the valley, glacio-fluvial outwash sands and gravels form the surface deposits. These deposits surround many drumlins. Deposits of unstratified glacial ground moraine, buried by outwash deposits, occur in the valley. Throughout the upland area, bedrock occasionally projects through the ground moraine mantle.

b. The carboniferous bedrock, through which the tunnel will be driven, is composed of two formations, locally known as the Worcester phyllite and the Oxford schist. The bulk of the return channel bedrock is gneiss, locally known as the Bolton gneiss. The intake structure will be founded in overburden which is underlain by the phyllite and the outlet works will be founded on the Oxford schist.

2-02. Location and Description of Site. - The site is located in the Towns of Auburn and Millbury, Massachusetts, approximately three miles southwest of Worcester. In the vicinity of the intake structure, the east slope of the Auburn valley rises quite abruptly to the rounded crest of Pakachoag Hill. The east flank of the hill, on which the outlet works will be located, has been quite thoroughly dissected by the tributaries of Hull Brook, a minor tributary of the Blackstone River. The return channel will be located in the Hull Brook valley. The site is partly cultivated fields adjoining residences and partly pasture land and is accessible by paved roads.

2-03. Subsurface Explorations. - Subsurface explorations were accomplished by core borings and seismic explorations. These explorations were made to locate and sample rock and to ddtermine the thickness and character of the overburden. The locations of the explorations are shown on Plates Nos. II-1, II-2, and II-3, "Plan and Profile of Subsurface Explorations and Rock Contours" and the records of the core borings on Plate No. II-4, "Record of Subsurface Explorations, No. 1."

a. Borings. - Foundation investigations to date have been accomplished by 23 borings, consisting of 692.8 feet of drive sample borings in overburden and 618 feet of rock cores. Exploration of the overburden has been carried out by cased drive samples. Rock samples were cored with a diamond bit. In the tunnel inlet and outlet areas, where the location and character of the bedrock is of important concern, rock penetration was carried well below the foundation grade of structure. With exception of two boreholes, drilling was done by contract. The bedrock encountered in holes at the above structures was pressure tested and equipped with observation well pipes to obtain records of ground water fluctuations. It is planned to obtain such records over a considerable period of time.

b. Seismic Explorations. - Sixty-six seismic lines, totaling 21,790 lineal feet, were explored in the vicinity of the proposed tunnel outlet works and along the proposed return channel. Seismic explorations were accomplished by refraction method to supplement available drilling data and thereby obtain information as to depths to firm rock in order to aid in selecting the most economical and desirable route of the return channel.

2-04. Geology of Site. - a. Tunnel. - (1) Overburden Conditions. - Except for some 500 feet at the intake end, the tunnel will be driven through the phyllite and schist of Pakachoag Hill, a rock-core hill covered by a mantle of compact, unstratified glacial ground moraine (moderately cohesive, impervious till). The tunnel section in earth will be driven through this structurally strong till. Bedrock is exposed in a rock cut on the west shoulder and near the crest of Pakachoag Hill adjacent to the tunnel alignment. Eastward from here, the till overburden thickens to a maximum of about 75 feet on the east slope of the hill.

(2) Stratigraphy and Structure. - Bedrock in the vicinity of the tunnel consists of two Carboniferous formations, the Worcester phyllite and the Oxford schist. The contact between the schist on the east and the phyllite to the west is obscured by overburden but from available information appears to strike northeast approximately through the crest of Pakachoag Hill. In general the bedrock surface east of the crest of the hill slopes toward the east; however, the prevailing easterly slope is interrupted by a series of low serrate ridges parallel to the strike of the formation. These ridges have moderate westward facing back-slopes, conforming to the prevailing foliation dip, while east slopes are nearly vertical -- the product of one member of the master joint system which parallels the strike of the formation.

(a) The Oxford schist, oldest of the two carboniferous formations, is exposed on the southeast shoulder of Pakachoag Hill approximately 0.5 mile from the proposed tunnel. Here there are a series of four low (10+ feet) parallel ridges, striking N. 30° E. with a voliation dip of 36° to the northwest. A parallel system of vertical master joints forms the east scarps of the outcrops. At the outcrop area, the schist is a lead-grey, quartz injection mica schist, containing a few small garnet metacrysts and some sericite.

(b) The Worcester phyllite has been encountered in borings and outcrops west of the crest of Pakachoag Hill. From west to east this formation varies from a soft black graphitic and carbonaceous phyllite in the vicinity of the intake structure to a black dense graphitic phyllite. The upper part of the last phase is quite soft and considerably broken. The dense black graphitic phase has been encountered as far east as BH-21 (see Plate II-1), which is approximately opposite the tunnel station at which rock tunneling will begin. East of here, the formation occurs as a dark grey, dense phyllite having a fine grain and containing some sericite. The grey phyllite contains many small quartz veins which closely follow the foliation in all its detail of small folds and crumples. Occasionally larger quarts veins are encountered which cut across the foliation. At nearby outcrops, the phyllite strikes approximately N. 30° E. and dips about 55° to the northwest. Unoriented foliation dips encountered in rock cores are of about the same magnitude. As previously indicated, the eastern limit of the phyllite is obscured by overburden and has not been closely defined by borings to date. However, it appears to be somewhat west of BH-2 (see Plate II-1), since at this location a mica schist was encountered.

b. Return Channel. - (1) Overburden Conditions. - Along the return channel the overburden character varies greatly. However, seismic explorations and some borings indicate that the bedrock surface is quite persistently mantled by glacial ground moraine (till) in which accumulations of boulders are frequent. The till appears at the ground surface to the vicinity of station 59+00. Downstream, the till is buried by increasing thicknesses of glacial lake bed deposits -- loose varved fine sand and sandy silt. Within limits of the channel, exploration data indicates that the lake bed deposits are abruptly terminated by an outcropping rock knob at station 110+00. Downstream from here the proposed return channel will be excavated in stratified glaciofluvial medium to coarse sand and gravel outwash deposits. The outwash deposits in the lower reach of the proposed return channel are locally masked by swamp deposits, whose thickness exceed 10 feet between stations 143+00 and 148+00.

(2) Structure and Stratigraphy. - The upper reaches of the return channel will cut through the previously described Oxford schist. However, the major portion of the return channel bedrock will consist of a gneiss and schist complex of undetermined age.

Several strips and lenses of schist of sedimentary origin are apparently infolded into the general complex. The rocks of these strips and lenses include brown biotite quartz schist and quartz-mica schist. The greater part of the bedrock exposures are a complex, closely squeezed mica gneiss - commonly being a medium-grained to fine-grained, brown, quartzose, biotite gneiss containing pinched quartz veins. The gneiss which will be excavated from the channel is a fairly sound rock and will be suitable for riprap.

(a) The bedrock surface, as determined by borings, seismic explorations, and field inspections, is quite irregular and probably contains a greater number of buried rock knobs than shown by the rock contours and profiles on Plates Nos. II-1, II-2, and II-3.

2-05. Geology of Drainage Area. - The drainage area will be formed in Kettle and Tatnuck brook valleys, above Leesville Pond, all of which are within the pre-glacial Auburn valley. Extensive glacial deposits of stratified outwash sand and gravel occur in these valleys, covering the bedrock to uncertain depth. Drumlins, composed of glacial till, occur in the major valley. The sides of the main valley are mantled with glacial ground moraine through which project occasional rock outcrops.

2-06. Foundation Conditions of Structures. - a. Intake Structure. It is tentatively planned to found intake structure on the structurally strong glacial till as preferable to the bedrock surface which a borehole at site of structure indicates occurs about 8 feet below the bottom of the proposed structure. Here the upper surface of the phyllite is somewhat fractured and might involve appreciable rock stripping to prepare for a structure foundation. A borehole about 50 feet from the proposed structure encountered graphitic phyllite whose upper 10 feet is fractured and quite soft, indicating that character of rock is very variable in this area.

(1) Explorations to date indicate that the bulk of the water in rock is carried in the upper fractured rock zone. If such is the case at the intake structure, water pressures in the rock at the side-hill site of the intake structure may require installation of relief wells into rock during construction to prevent loosening overburden during structure excavation. Observations are being taken at the proposed intake structure to further investigate these foundation conditions.

b. Outlet Portal will be founded on the Oxford schist which has ample strength to support the proposed structure. The tunnel outlet location has been chosen to give 20 feet rock cover over outlet portal and may be revised later when rock surface has been better determined.

2-07. Foundation Problems and Proposed Treatments. - a. Tunnel It is not anticipated that any major problem will be encountered which cannot be handled by ordinary tunnel driving methods.

(1) Moderate local rock-sealing, especially of roof-rock, is anticipated throughout length of the tunnel. It is believed that scaling can be generally controlled by applying spray coats of cement mortar.

(2) It is believed that timber support can be held to a minimum (possibly 5% of the total rock tunnel will require timber support) and that the bulk of such support will be confined to that section of tunnel driven through the phyllite.

(3) Since the tunnel orientation is favorable by being nearly perpendicular to the strike of the formations, overbreak will probably be moderate. Although study of surface outcrops and rock cores do not indicate presence of crushed zones and/or areas of close jointing, it is conceivable that such zones of weakness do occur; in fact, similar zones have been seen in the Oxford schist approximately 3 miles southwest of the site in a deep rock cut.

b. Effect on Existing Wells. - It is anticipated that temporary drawdown of existing water supply wells along the tunnel may be the most serious minor problem encountered. There are a considerable number of existing wells now in use in region of the tunnel as shown on Plate No. II-5 and described in Table No. II-1.

(1) It will probably require about a year to drive the tunnel and an additional three months to complete the lining -- a period of some 15 months during which ground water in the rock will find easy exit through the tunnel. Following completion of tunnel lining, it is estimated that it may require another year for ground water to resume normalcy -- provided concrete lining and grouting planned is successful in preventing tunnel from leaking considerably and thus permanently lowering the ground water.

(2) Water supply wells in vicinity of tunnel are of two types: shallow dug wells in overburden and deep driven wells in rock.

(a) Dug Wells. - There are seven shallow dug wells within zone of drawdown from tunneling. All of these wells are founded in impervious till and normally would not be subject to appreciable drawdown except over a long time. With one exception, all of these wells are located near the intake end of the tunnel where it is probable that contractor will elect to drive overburden section of tunnel as a first operation -- facilitating early erection of intake structure -- leaving tunnel in earth supported by temporary timbering. If such is accomplished, the unlined tunnel will act as a large horizontal drain and complete drawdown of these wells during construction is probable. A shallow dug well near the outlet portal was adversely affected temporarily by blasting during drilling of a nearby foundation borehole. Thus, it is anticipated that drawdown of this well may well occur during tunneling.

(b) Deep Driven Wells in Rock. - There are 17 water supply wells driven into rock within possible zone of drawdown by tunneling. Although these wells are bottomed some 60-120 feet above proposed tunnel, marked temporary drawdown of these wells is anticipated if same water-carrying rock joints feeding wells are also intersected by the tunnel. Direct communication is probable via the master joint system which strikes approximately perpendicular to the tunnel.

(3) A survey of existing water supply wells has been made to compile data to study methods of supplying water temporarily in event certain wells should be adversely affected by tunneling. A moderate number of observation wells are now being installed. From these and existing foundation boreholes equipped for ground water observation, it is hoped to obtain records of ground water fluctuations over a period of at least one year prior to construction in order to forestall unwarranted claims.

(4) Insofar as artesian pressure does not seem to occur except locally on the lower slopes of the hill, it is anticipated that whatever water enters through joints and cracks will readily drain out through tunnel without inconveniencing the contractor provided tunnel is driven upgrade.

2-08. Sources of Construction Material. - A detailed account of construction materials will be found in Appendix VIII, "Availability of Construction Materials."

2-09. Conclusions - Geologic Feasibility of Site. - The overall geologic conditions are favorable for the construction of the proposed structures.

RECORD OF EXISTING WELLS

PAKACHOAG HILL

WORCESTER DIVERSION

26 March 1946
Soils Laboratory

Number Designation Shown on Plan (A)	Well Owner	Location of Well	Well Data: Type	Diameter Inches	Depth – Feet: Total	Dug	Driven	Pump Type	Availability for Observation	Performance Record	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(1)	Northern limit of Auburn municipal water main on Pakachoag Hill										
(2)	Albert E. Moon	307 Pakachoag St.	Driven	6	124	0	124	Electric	Pump plunger prevents observation	"No Failures"	Sole source of water. Well driven entirely in rock.
(3)	James E. Fish	298 Pakachoag St.	"	6	90±	0	90±	"	Pump plunger prevents observation	" "	Sole source of water. Well top buried 5'± in back yard.
(4)	Hallis A. Pratt	291 Pakachoag St.	Combination	6 & 18	85	17.5	67.5	Hand & Electric	Pump plunger prevents observation	"No failures in 35 yrs."	Sole source of water for this house and next house north.
(5)	Worcester Rendering Co.	286 Pakachoag St.	Combination	6 & 18	65-70±	20-25±	45-50±	"	Plant Supt. not prepared to state	Unknown	Sole source of water for two-family house. Data obtained from Arthur Eaton.
(6)	Hallis A. Pratt	289 Pakachoag St.	Combination	See data above - H. Pratt -- House occupied by A. T. Deacon							
(7)	Mrs. Prescilla Larrabee (House occupied by A. S. Coldwell)	281 Pakachoag St.	Driven	6	42	0	42	Electric	Plunger prevents observation	"No failures in 30 yrs."	---------
(8)	Patrick Rooney	224 Pakachoag St.	"	6 ?	50±	0	50±	"	Owner would object	Unknown	Municipal water piped to cellar wall but not into house.
(9)	Arthur Eaton	222 Pakachoag St.	Combination	6	45-50±	20-25	45-50±	"	Owner would not object but pump plunger would probably prevent observation	"No failures"	Driven into rock 25'±. Municipal water piped to cellar wall but not into house.
(10)	J. A. MacLennan	216 Pakachoag St.	No information available -- Well is bricked up							Unknown	---------
(11)	A. J. Cutting	104-106 Pakachoag St.	Driven	6	120-130	0	120-130	Electric	Owner would object	Unknown (b)	Four wells have been drilled at this farm, of which two were abandoned and backfilled. City water available at house and barn.
(12)	A. J. Cutting (Barn)	Opposite 104-106 Pakachoag St.	"	6	120-130	0	120-130	"	" " "	(c)	
(13)	A. J. Cutting, (Cottage)	104-106 Pakachoag St.	"	6	120-130	0	120-130	"	" " "	Unknown (b)	
(14)	---------	196 Pakachoag St.	There is no well at this house				----	-----	-----------	--------	---------------
(15)	Andrew Love	9 Curtis Street	Combination	6	118	10	108	Electric	Owner states, "Well not in use. Observations can be made."	"Well generally dry"	Only house on Curtis St. where city water available.
(16)	William Baisley	71 Curtis Street	Combination	6 & 24±	85	30±	55±	Hand & Electric	Well can be observed	"No failures"	---------
(17)	W. Q. Devio	80 Curtis Street	Dug	18	22±	22±	0	Electric	Well can be observed	"Mostly dry over period of 2 months in summer of 1945. No previous failures."	Blasting to advance casing at BH-5 caused well to go dry temporarily.
(18)	Charles King	103 Elmwood St.	Driven	6	78	0	78	Electric	Pump plunger prevents observation	"No failures"	Only house on Elmwood St. dependent on well water. City water main in street.
(19)	--------	There is no well here (166 Hampton Street, house occupied by R. H. Whitcomb, owned by H. A. Davis)								---------	Only house on Hampton St. having city water.
(20)	M. Price	221 Hampton St.	Driven	6	70	0	70	Electric	Can not be observed	Unknown	Drilled entirely in rock. Data from former owner, A. Eaton.
(21)	Hylas Wolcott	226 Hampton St.	"	6	235 ?	0	235 ?	Electric	Can not be observed	"No failures since 1928."	Depth of well probably erroneous. Now drawing water from depth of 45'.
(22)	---------	Eaton Avenue	This house dependent upon neighbors for water -- carrying water in pails.							---------	---------
(23)	George Loussier	45 Eaton Avenue	Dug	30	19	19	0	Electric	Owner would object	"Failed every year except last."	---------
(24)	G. Duchasne	11 Eaton Avenue	(No satisfactory information available)					Electric	Owner would object	---------	---------
(25)	E. J. LaCroix	9 Eaton Avenue	Driven	6	Unknown	0	Unknown	Electric	Owner would object	Unknown	Owner unable to supply data.
(26)	C. J. Chalifoux	3 Eaton Avenue	Dug	18	24	24	0	Electric	Owner would object	"No failures in 17 yrs."	---------
(27)	Thomas Hazeldine	6 Eaton Avenue	Dug	18	15	15	0	Electric	Owner would object	"No failures"	---------
(28)	Mrs. Camille Fornari	10 Eaton Avenue	Dug	18	15	15	0	Electric	Possibly could observe one of these wells	Unknown	Owner unavailable. Data from T. Hazeldine.
(28)	" " "	" " "	"	18	10	10	0	?			
(29)	George A. LaRoche (Glenn Tice)	18 Eaton Avenue	(Mr. Tice obtains water from LaRoche well)					Electric	-------------	---------	---------
(30)	George LaRoche	18 Eaton Avenue	Combination	6 & 18	110±	10±	100±	Electric	Owner would object	No failures since dug well. Extended by driven well.	---------
BH-3	U.S.E.D.	See plan	Drilled	3-1/2	96.1	0	96.1	None	Casing left in hole	Continuous artesian flow over top of casing (2' stickup)	---------
BH-4	"	" "	"	"	100	0	100	"	O.W. installed to 18' in overburden	Not applicable	----------
BH-5	"	" "	"	"	91.2	0	91.2	"	-----------	" "	----------
BH-7	"	" "	"	"	80	0	80	"	O.W. installed to 18' in overburden	" "	----------
BH-15	"	" "	"	"	76	0	76	"	O.W. installed to 18' in overburden	" "	----------
BH-19	"	" "	"	"	118.7	0	118.7	"	O.W. installed to 58' in overburden	" "	----------
BH-20	"	" "	"	"	34.7	0	34.7	"	O.W. installed to 8' in overburden	" "	----------
BH-21	"	" "	"	"	109.0	0	109.0	"	O.W. installed to 17' in overburden	" "	----------

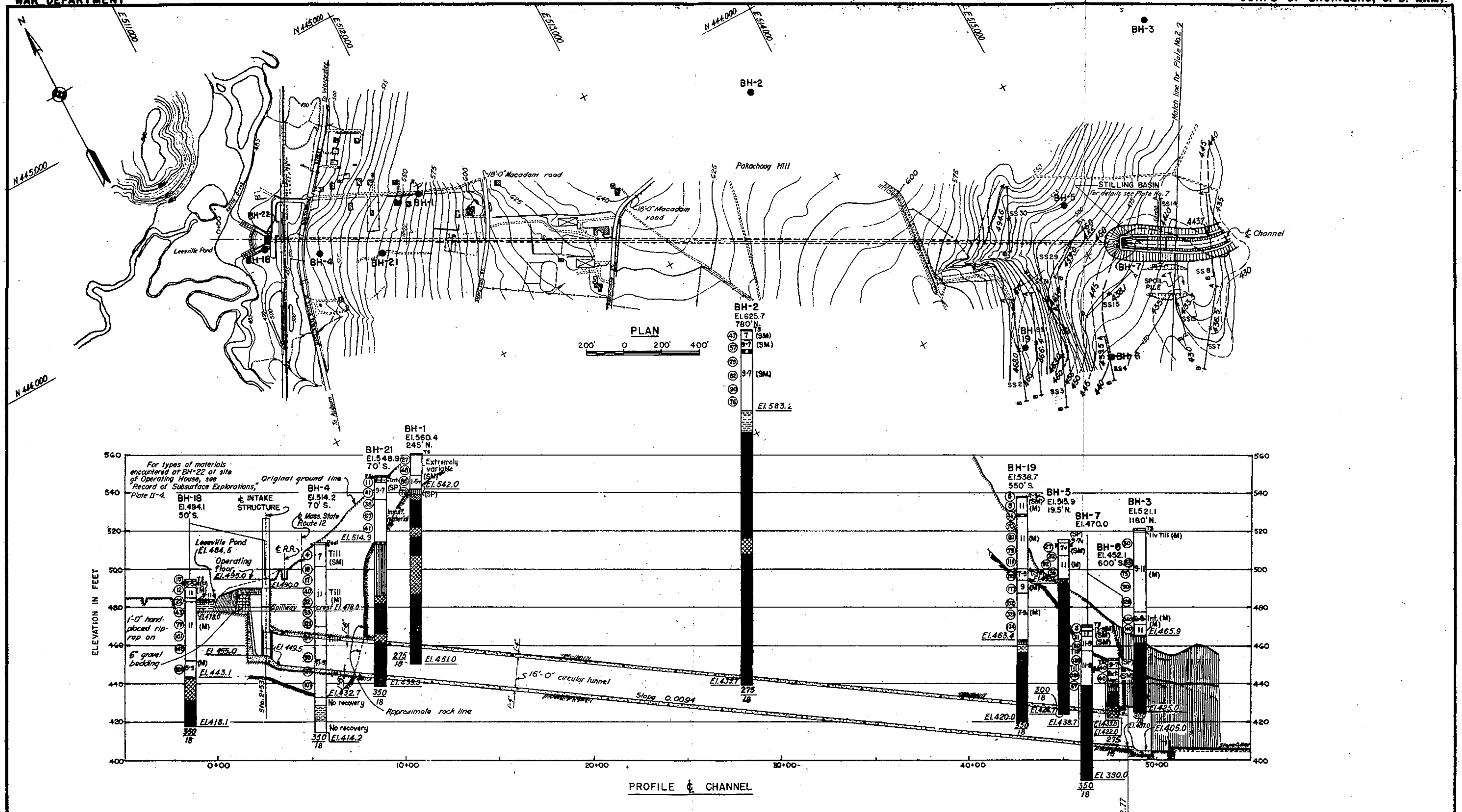
NOTES:

(a) Plan entitled "Worcester Diversion, Existing Wells."
(b) This well supplies 6 families, at least part time.
(c) It was necessary to use city water for watering stock for period of 3 months during summer of 1944. This suggests either temporary failure of well or that water was very low in well during this period.

APPENDIX II TABLE NO. I

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTES

For general notes, legend and description of numerical soil classes, see sheet of "Record of Subsurface Explorations."

For Seismic legend, see Sheet No. 2.

KEY	DATE	REVISION (indicated by △)	REV.BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

PLAN & PROFILE OF SUBSURFACE EXPLORATIONS AND ROCK CONTOURS

BLACKSTONE RIVER MASSACHUSETTS

IN SHEETS 200' 0 200' 400' SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: SENIOR ENGINEER HEAD, SOILS SECTION — APPROVAL RECOMMENDED: HEAD ENGINEER CHIEF, ENGINEERING DIV. — APPROVED: COL. CORPS OF ENGINEERS DISTRICT ENGINEER

PREPARED: SOILS SECTION — DRAWN: A.D.C.W. TRACED: CHECKED:

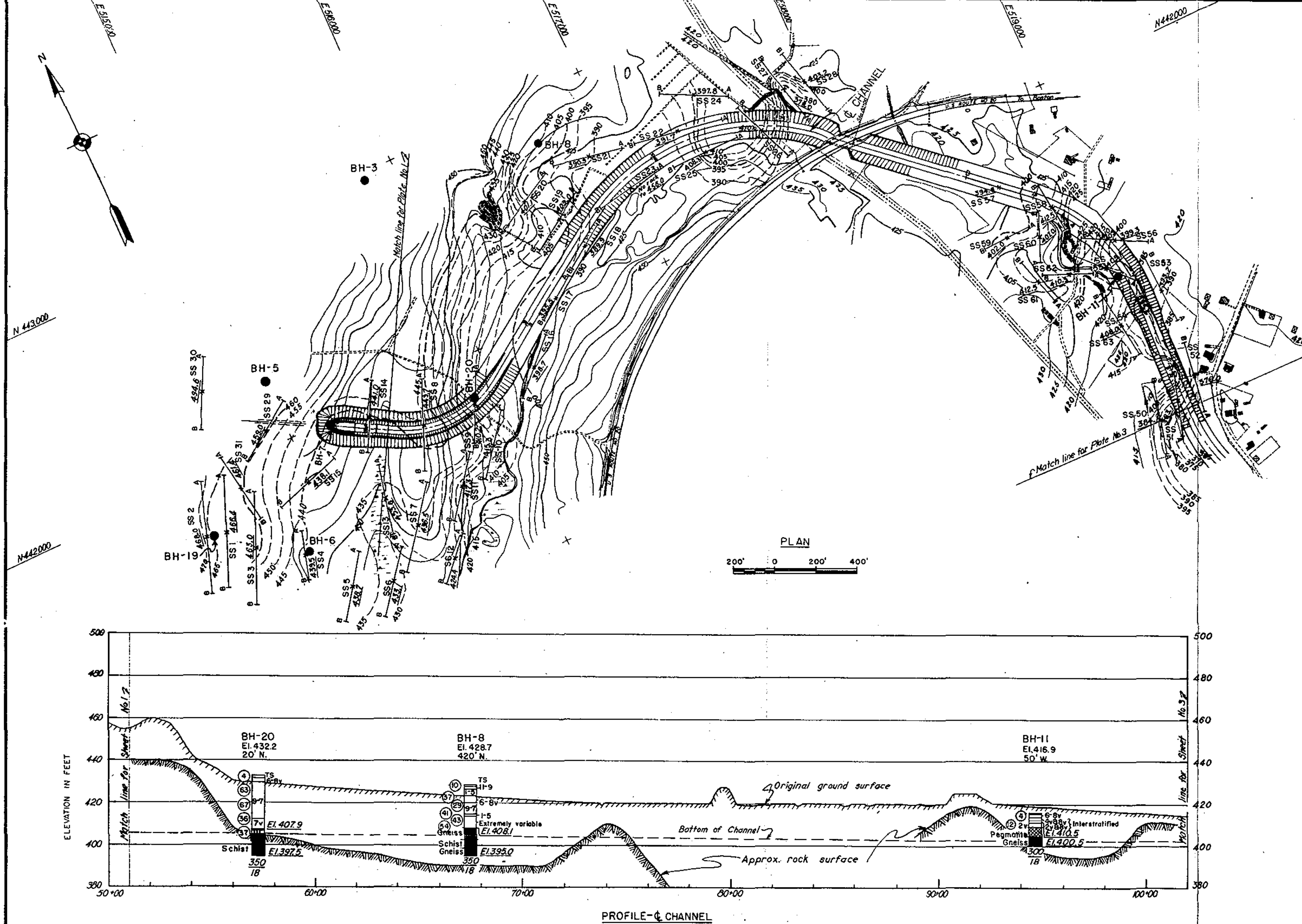
SL NO. WDB-A3

FILE NO. BE-2-1007

APPENDIX II PLATE NO. 1

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



LEGEND

● BH - Drive sample boring - overburden sampled by driving spoon and bedrock cored by diamond bit.

Bedrock outcrop.

Earth contours.

Rock contours.

A - Ahead end of seismic line

B - Back end of seismic line

Seismic line

SS 59 - Seismic number

Visual Estimate of Permeability of Soils

(P)	Pervious	$10-500 \times 10^{-4}$ cm/sec.
(SP)	Semi pervious	1-10 do
(RM)	Semi-impervious	0.1-1 do
(M)	Impervious	< 0.1 do

NOTES

Elevations refer to Mean Sea Level Datum.

Contour interval is 5 feet.

Underlined figures on plan are elevations at top of solid rock, determined by Seismic Method at center of seismic lines.

For a more complete legend, and description of numerical soil classes, see "Record of Subsurface Explorations."

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

PLAN & PROFILE OF SUBSURFACE EXPLORATIONS AND ROCK CONTOURS

BLACKSTONE RIVER MASSACHUSETTS

IN SHEETS 200' 0 200' 400' SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: K. L. Lane, SENIOR ENGINEER, HEAD SOILS SECTION

APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV.

APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: R. G. Abbott, SOILS SECTION

DRAWN: A. B. C. W.

TRACED:

CHECKED:

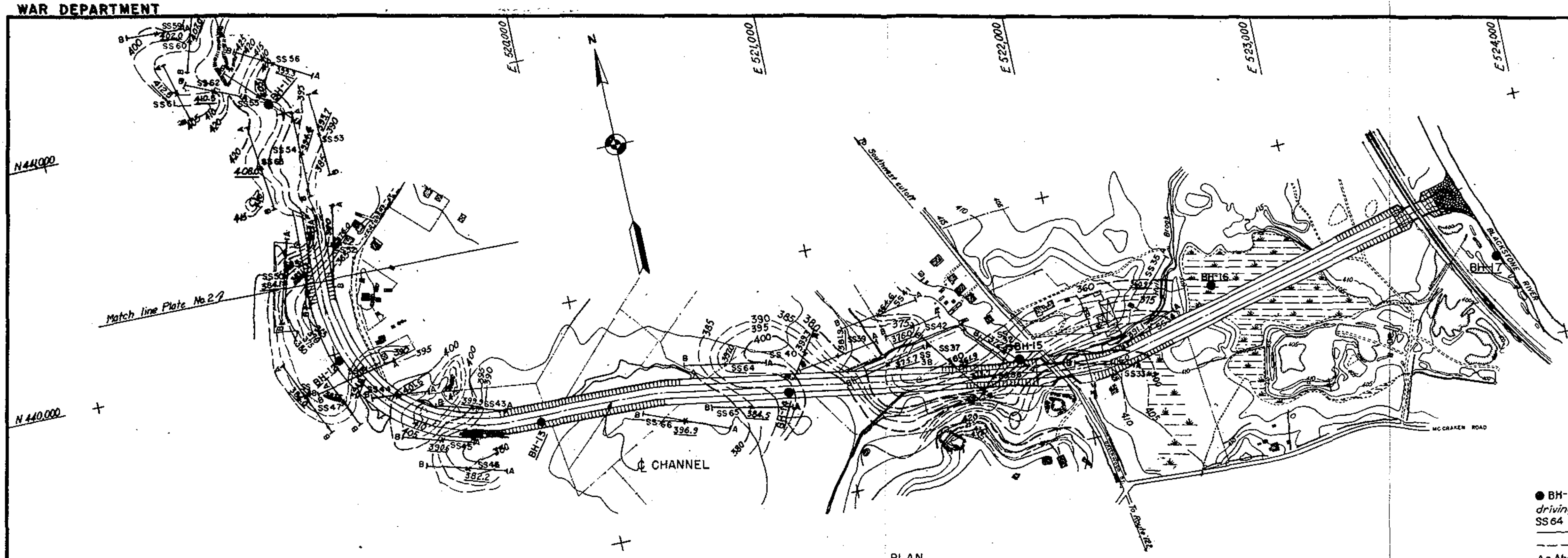
SL NO. WDB-A4

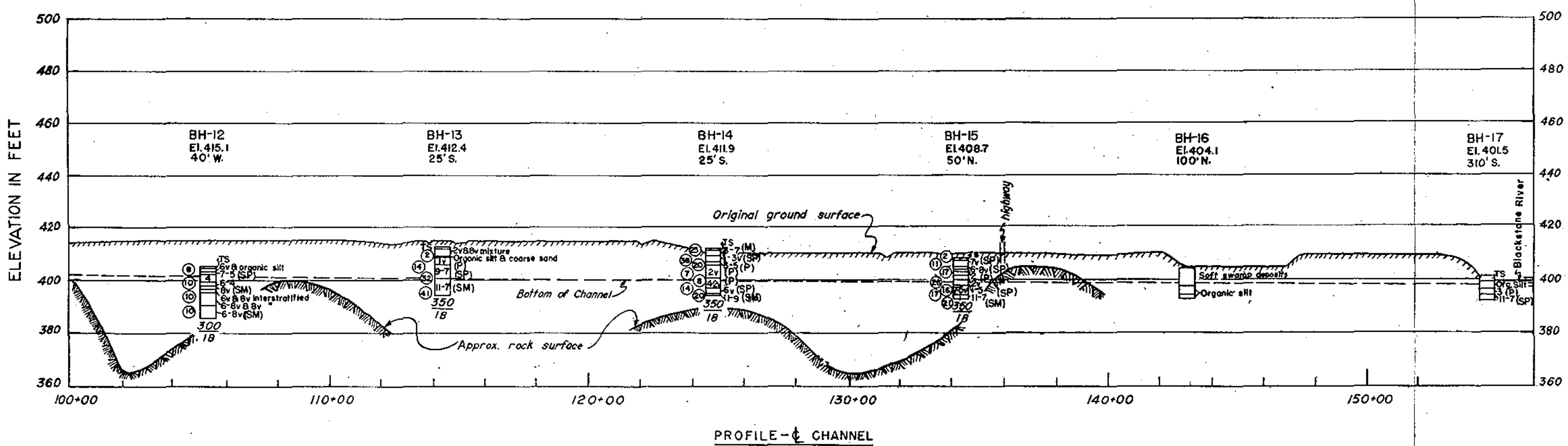
FILE NO. BE-2-1008

APPENDIX II PLATE NO. 2

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY





LEGEND

● BH- Drive sample boring - overburden sampled by driving spoon and bedrock cored by diamond bit.
SS 64 - Seismic line number.
——— Earth contours
— — — Rock contours
A - Ahead end of seismic line
B - Back end of seismic line
Seismic line
Bedrock outcrop

Visual Estimate of Permeability of Soils

(P)	Pervious	10 - 500 x 10^{-4} cm/sec.
(RP)	Random pervious	1 - 10 do
(RM)	Random impervious	0.1 - 1 do
(M)	Impervious	< 0.1 do

NOTES

Elevations refer to Mean Sea Level Datum.
Contour interval is 5 feet.
Underlined figures on plan are elevations at top of solid rock, determined by Seismic Method at center of seismic lines.
For a more complete legend, and the description of numerical soil classes, see "Record of Subsurface Explorations."

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL
WORCESTER DIVERSION
PLAN & PROFILE OF SUBSURFACE EXPLORATIONS AND ROCK CONTOURS
BLACKSTONE RIVER MASSACHUSETTS

IN SHEETS 200' 0 200' 400' SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

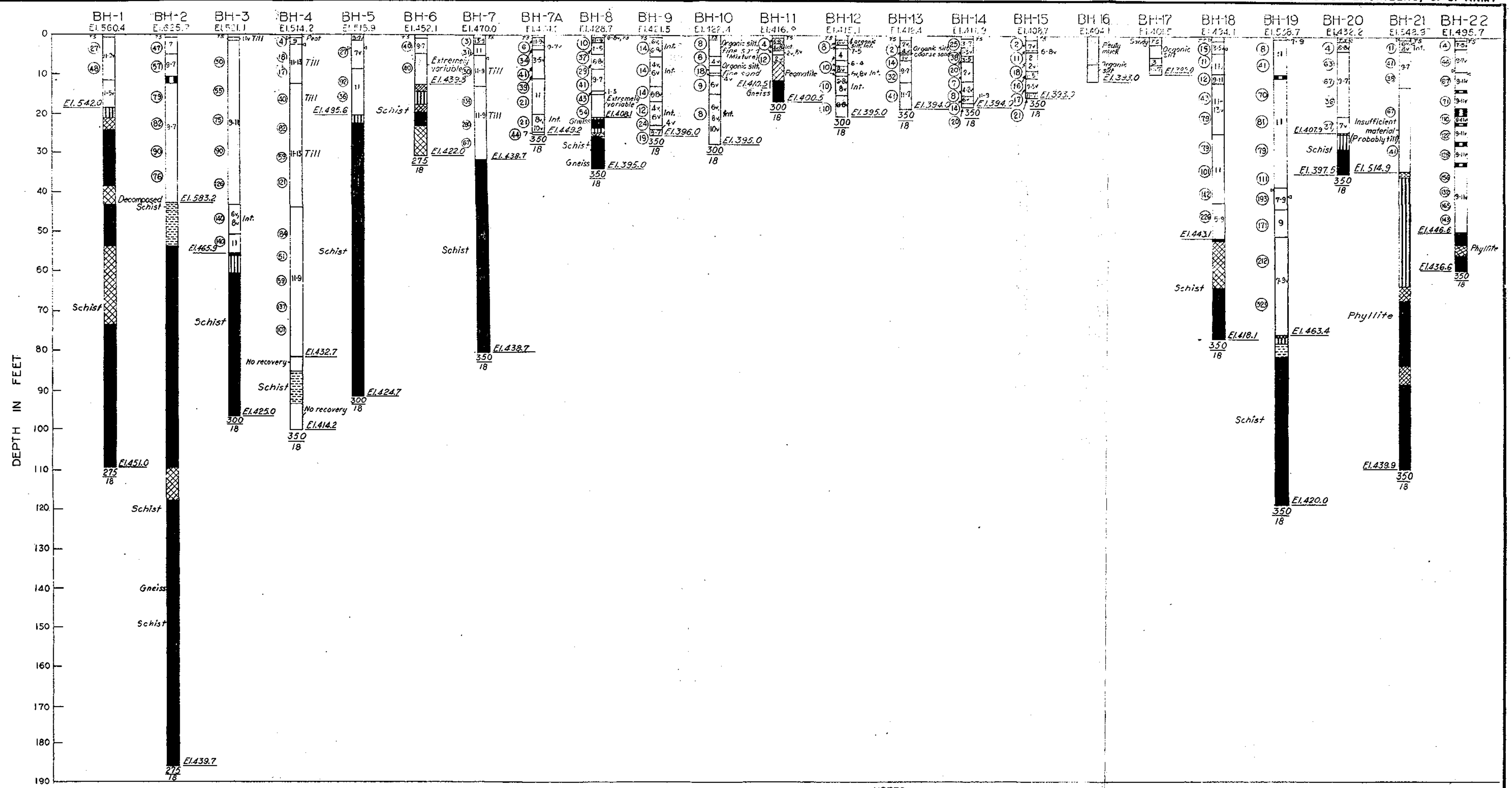
SUBMITTED: K. S. Lane, SENIOR ENGINEER HEAD, SOILS SECTION
APPROVAL RECOMMENDED: R. H. Hayes, HEAD ENGINEER CHIEF, ENGINEERING DIV.
APPROVED: COL., CORPS OF ENGINEERS DISTRICT ENGINEER

PREPARED: R. E. Abbott, SOILS SECTION
DRAWN: A. B. C. W.
TRACED:
CHECKED:
SL NO. WDB-A5
FILE NO. BE-2-1009

APPENDIX II PLATE NO. 3

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



DESCRIPTION OF SOIL CLASSES

1. *Graded from Gravel to Coarse Sand* - Contains little medium sand.
2. *Coarse to Medium Sand* - Contains little gravel and fine sand.
3. *Graded from Gravel to Medium Sand* - Contains little fine sand.
4. *Medium to Fine Sand* - Contains little coarse sand and coarse silt.
5. *Graded from Gravel to Fine Sand* - Contains little coarse silt.
6. *Fine Sand to Coarse Silt* - Contains little medium sand and medium silt.
7. *Graded from Gravel to Coarse Silt* - Contains little medium silt.
8. *Coarse to Medium Silt* - Contains little fine sand and fine silt.
9. *Graded from Gravel to Medium Silt* - Contains little fine silt.
10. *Medium to Fine Silt* - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.

10C. *Medium Silt to Coarse Clay* - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.

11. *Graded from Gravel or Coarse Sand to Fine Silt* - Contains little coarse clay.
12. *Fine Silt to Clay* - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of clay.

12C. *Clay* - Contains little silt. Possesses behavior characteristics of clay.

13. *Graded from Coarse Sand to Clay*. Contains little fine clay (colloids). Possesses behavior characteristics of silt.

13C. *Clay* - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

LEGEND

BH - Drive sample boring - overburden sampled by driving spoon and bedrock cored by diamond bit.

TS Topsoil

6 Providence Classification - grain size number.

6v Providence Classification - grain size number - Visual classification.

Ground water level range during period of observation. (Readings intermittent.)

(18) Penetration: No. of blows per ft. penetration of 275-350 lb. hammer falling 18" on sampling spoon.

4v 6v 8v Int. Interstratified

Boulder

Rock core recovery 0-15%.

Rock core recovery 16%-50%.

Rock core recovery 51%-75%.

Rock core recovery 76%-100%.

350 - Weight of hammer in lbs.

18 - Inches, hammer falls on sampling spoon.

NOTES

Elevations refer to Mean Sea Level Datum.

For locations of borings, see Sheet No. 1 for this site.

The logs, samples and test results pertaining to the above investigations may be inspected at the U.S. Engineer Office, Providence, R.I.

Double class numbers (as 6-4) indicate soils grading between two classes, as a soil where coarser fraction follows gradation of initial class (6) and finer fraction follows gradation of final class (4). Higher followed by a lower class number indicates more uniform gradation (as 6-4) and lower followed by a higher class number indicates more variable gradation (as 2-4) than that of a class 4 soil.

The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the government will not be responsible for any deduction, interpretation, or conclusion therefrom made by any bidder or Contractor.

KEY	DATE	REVISION (Indicated by △)	REV.BY	CK.BY	AP.BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

RECORD OF SUBSURFACE EXPLORATIONS NO. 1

BLACKSTONE RIVER MASSACHUSETTS

IN 3 SHEETS | SCALE AS SHOWN | SHEET NO. 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JUNE 1946

SUBMITTED: K S Lane — SENIOR ENGINEER, HEAD, SOILS SECTION

APPROVAL RECOMMENDED: [signature] — HEAD ENGINEER, CHIEF, ENGINEERING DIV.

APPROVED: [signature] — COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: SOILS SECTION

DRAWN: A.B.C.W.

TRACED:

CHECKED:

SL NO. WDB-A2

FILE NO. BE-2-1003

APPENDIX II PLATE NO. 4

LEGEND

① *Number refers to place where well and/or municipal water supply data was obtained. See Table entitled "Record of Existing Wells, Pakachoag Hill." for description.*

● BH - *Bore hole exploration by U. S. E. D. having existing Observation Well, except as noted.*

Overburden Observation Well - Initial installation.

Rock Observaton Well - Initial installation.

Overburden Observation Well - Installed later, during construction.

Rock Observation Well - Installed later, during construction.

Bedrock outcrop.

Hydrant.

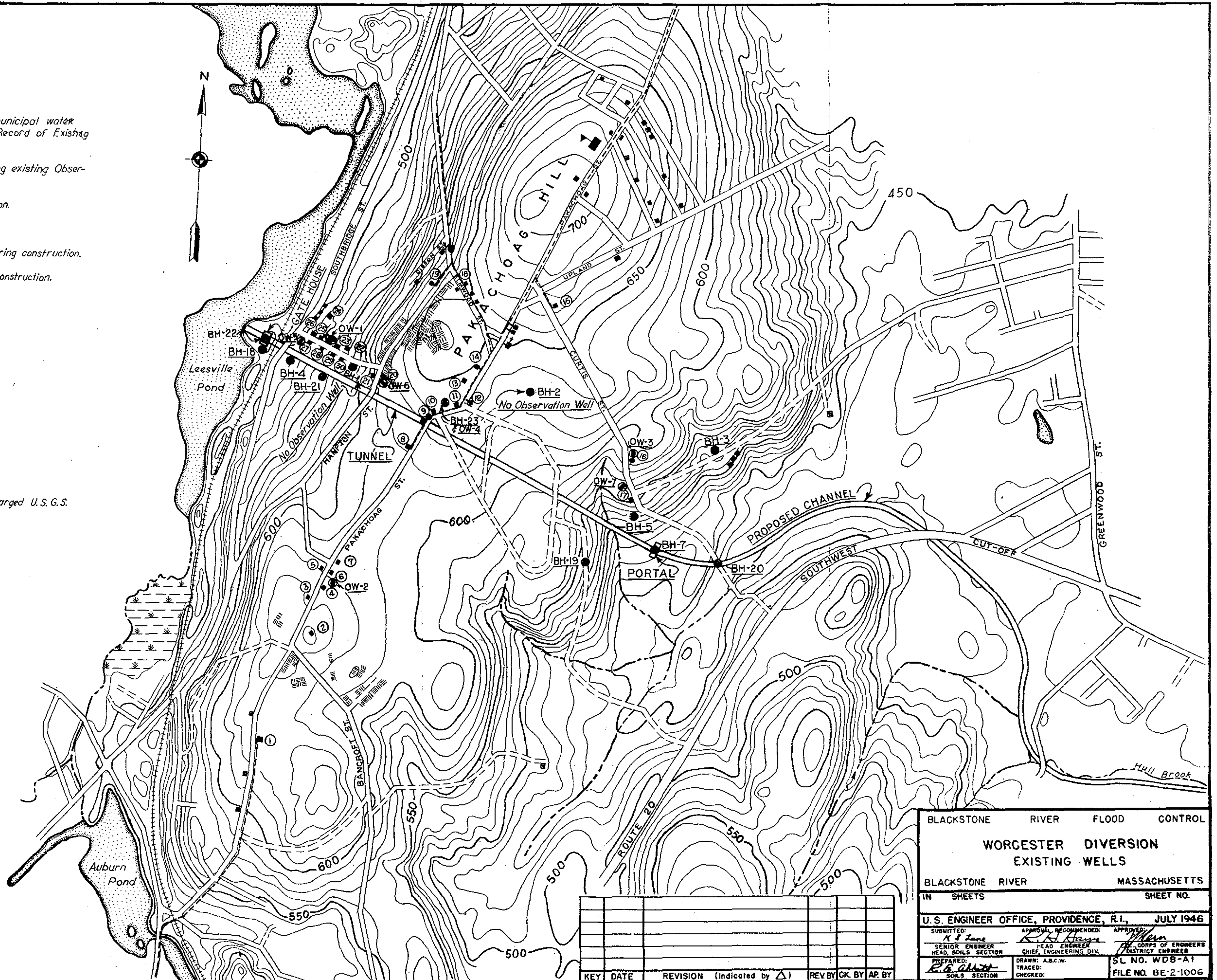
Water main.

NOTES

Topographic and cultural data taken from enlarged U. S. G. S. Quadrangle.

Contour interval is 10 feet.

Elevations refer to Mean Sea Level Datum.



KEY	DATE	REVISION (indicated by △)	REV. BY	CK. BY	AP. BY

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

EXISTING WELLS

BLACKSTONE RIVER MASSACHUSETTS

IN SHEETS SHEET NO.

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JULY 1946

SUBMITTED: K. S. Lane, SENIOR ENGINEER HEAD, SOILS SECTION

APPROVAL RECOMMENDED: HEAD ENGINEER CHIEF, ENGINEERING DIV.

APPROVED: CORPS OF ENGINEERS DISTRICT ENGINEER

PREPARED: SOILS SECTION

DRAWN: A.B.C.W.
TRACED:
CHECKED:

SL NO. WDB-A1

FILE NO. BE-2-1006

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX III

SOIL DATA AND ANALYSIS

To Accompany Definite Project Report
dated Sept. 1946

WORCESTER DIVERSION

APPENDIX III. SOIL DATA AND ANALYSIS

CONTENTS

TABLES

PLATES

APPENDIX III - SOIL DATA AND ANALYSIS

3-01. Laboratory Test Methods. - a. Soil Classification. - (1) The Providence District Soil Classification was used wherein soils are described by a name, covering general type of soil, plus a grain size number which is a function of position and shape of the grain size curve. The 16 classes of grain size numbers are shown graphically on Plate No. III-1 and described in Table III-1. Grain size numbers are determined from results of grain size tests, but to a major extent such is accomplished from visual examination in which case subscript "v" is included. Use of double class number as (4-2) indicates that the coarser portion of the grain size curve lies in the Class 4 range and the finer in the Class 2 range - in this case a very uniform sand. In contrast, Class 2-4 represents a much more variably graded sand. In using double class numbers, even numbers (uniform gradation) are never used in combination with odd numbers (variable gradation).

(2) Where pertinent to design problems, visual permeability estimates are made during soil classification to assign materials into the following permeability groups. For foundation soils, classification is made for estimated or measured density in natural deposit.

Permeability Group	Coefficient of Permeability (x 10^{-4} cms/sec)
Impervious (M)	0.1
Semi-impervious (SM)	0.1 - 1.0
Semi-pervious (SP)	1.0 - 10
Pervious (P)	10 - 500
Very Pervious (VP)	500

b. Grain Size. - Grain-size curves of samples were obtained by sieve and hydrometer analysis of representative samples for the major soil types encountered.

c. Water Content and Natural Density. - Water contents and natural densities were determined with reasonable approximation from short soil cylinders paraffined when taken as boring drive samples.

d. Other Tests. - Tests for shear, consolidation and permeability were not performed in the Definite Project stage of the investigation.

3-02. Foundations Conditions - Intake Channel. - The soil mantle at the intake is compact glacial till (classes 9 and 11), structurally strong and impervious. For such a compact foundation, no appreciable settlement is anticipated for the intake structure and cofferdamming over the impervious till should not offer a particular problem. A centerline soils profile is shown on Plate No. II-1.

3-03. Foundations Conditions and Stability - Return Channel. - The channel will be excavated mainly in soil overburden, but some rock excavation will be necessary. Centerline soil profile is shown on Plates Nos.

II-2 and II-3. Gradation curves for typical soils are shown on Plate No. III-2.

a. Outlet Portal Area. - The channel will be excavated in bedrock with till overburden. Progressing down the channel below the outlet portal the channel cut leaves the bedrock and will be entirely in till. The till is structurally strong and slopes greater than 1 on 2-1/2 would be stable; however, a 1 on 2-1/2 slope has been selected to facilitate power mowing of the portion of the slope to be turfed and for uniformity with the channel immediately downstream.

b. Lake Bed Reach. - At point where channel joins Hulls Brook to approximately Station 109+00 the channel will be excavated in former glacial lake bed deposits of varved silts and fine sands (classes 6, 6-8, 8 and 10). Gradation of these types of soils forming the individual varves can be judged from Plate No. III-1, it not being customary to conduct grain size tests on disturbed or mixed samples of a varved deposit. These soils are structurally weak (loose natural condition and low permeability), are easily eroded and are strongly frost acting.

(1) Although no shearing strength data were obtained from soil samples (samples being too small in diameter), values were estimated from similar materials tested for other sites. Based on assumed values of $\phi = 31°$, $c = 0.05$ tons per sq. ft., a slight factor of safety against a slide on sudden drawdown of design discharge in the channel is provided by slopes of 1 on 2-1/2. Some small local slides may occur on drawdown because of local zones of weaker material; but, it has not been considered economically worthwhile to obtain complete safety against such.

(2) For maintaining stability of the slopes, the design requires spoil piles of excavated soils be given a slope not greater than 1 on 3 parallel to the channel and that a berm be provided between top of channel cut and toe of spoil pile.

(3) The fine soils of the lake bed area are highly frost heaving; and, therefore, unprotected slopes are markedly subject to frost sloughing during spring thaw. From experience at other sites, covering the slope with a filter blanket of appreciably more pervious material has been quite successful in resisting frost sloughing and such is, therefore, included in this design (see Appendix V).

c. Sand and Gravel Reach. - Between approximate stations 110 to 143, the channel will be in medium to coarse sands and gravels. These soils are sufficiently strong to permit slope steepening; but, the slope of 1 on 2-1/2 has been continued for uniformity and to facilitate power mowing of turf.

d. Swamp Reach. - From Station 143 to Station 148 (approximately) the channel will be shallow, 7-8 feet deep, and will be excavated through a swamp containing peat and organic silt. This material has little structural strength, but the cut being so shallow, no special treatment has been considered justified.

e. <u>Outlet at Blackstone River.</u> - Between Station 148 and the Blackstone River, the channel will be excavated through sands and gravels. A small amount of organic silt will probably be encountered in the river bank.

TABLE I

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

PROVIDENCE DISTRICT SOIL CLASSIFICATION

CLASS	DESCRIPTION OF MATERIAL
1	Graded from Gravel to Coarse Sand.– Contains little medium sand.
2	Coarse to Medium Sand.– Contains little gravel and fine sand.
3	Graded from Gravel to Medium Sand.– Contains little fine sand.
4	Medium to Fine Sand.– Contains little coarse sand and coarse silt.
5	Graded from Gravel to Fine Sand.– Contains little coarse silt.
6	Fine Sand to Coarse Silt.– Contains little medium sand and medium silt.
7	Graded from Gravel to Coarse Silt.– Contains little medium silt.
8	Coarse to Medium Silt.– Contains little fine sand and fine silt.
9	Graded from Gravel to Medium Silt.– Contains little fine silt.
10	Medium to Fine Silt.– Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10C	Medium Silt to Coarse Clay.– Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Graded from Gravel or Coarse Sand to Fine Silt.– Contains little coarse clay.
12	Fine Silt to Clay.– Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12 C	Clay.– Contains little silt. Possesses behavior characteristics of clay.
13	Graded from Coarse Sand to Clay.– Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13C	Clay.– Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

ENGINEERING DIVISION – SOILS LABORATORY PROVIDENCE, R. I.

PROVIDENCE DISTRICT SOIL CLASSIFICATION

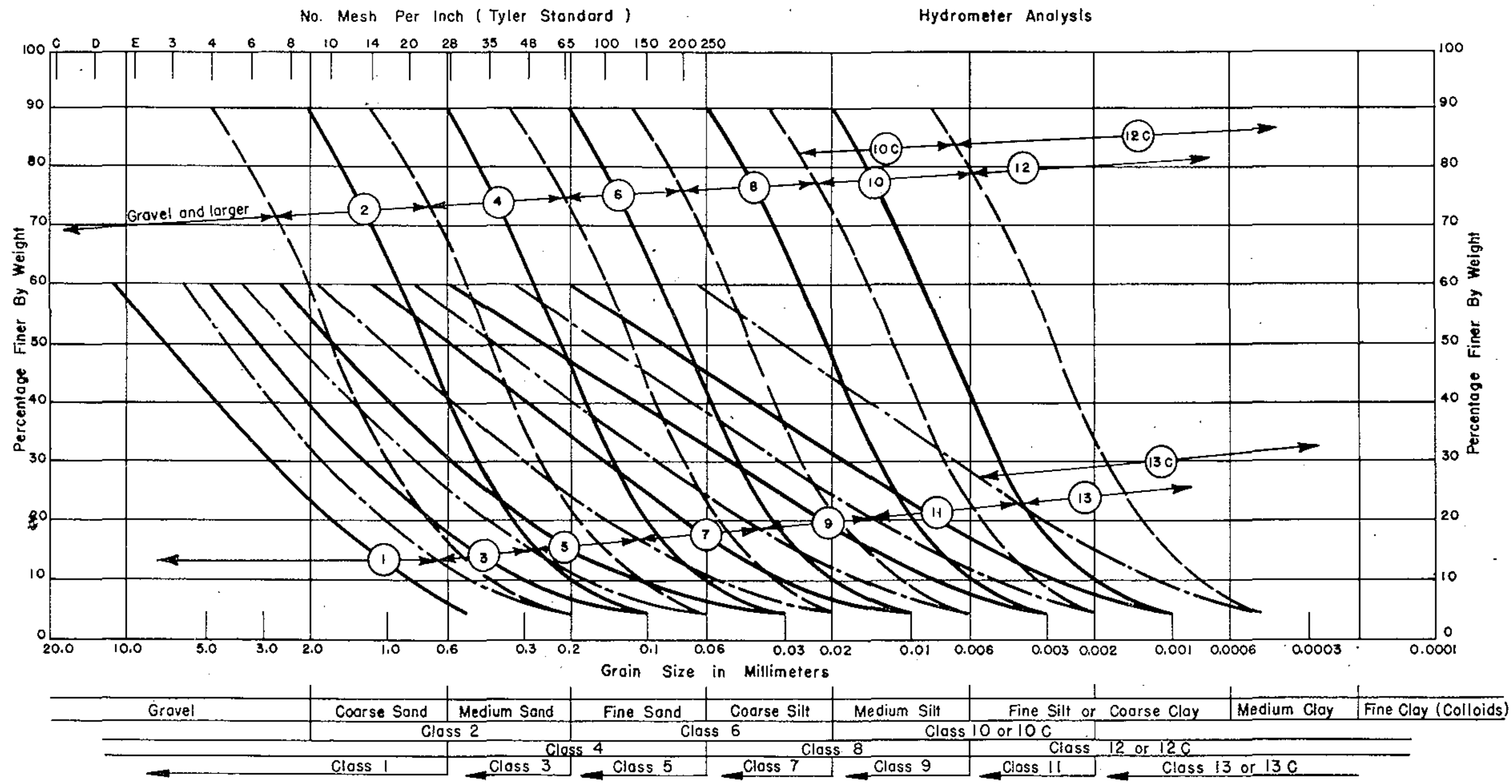
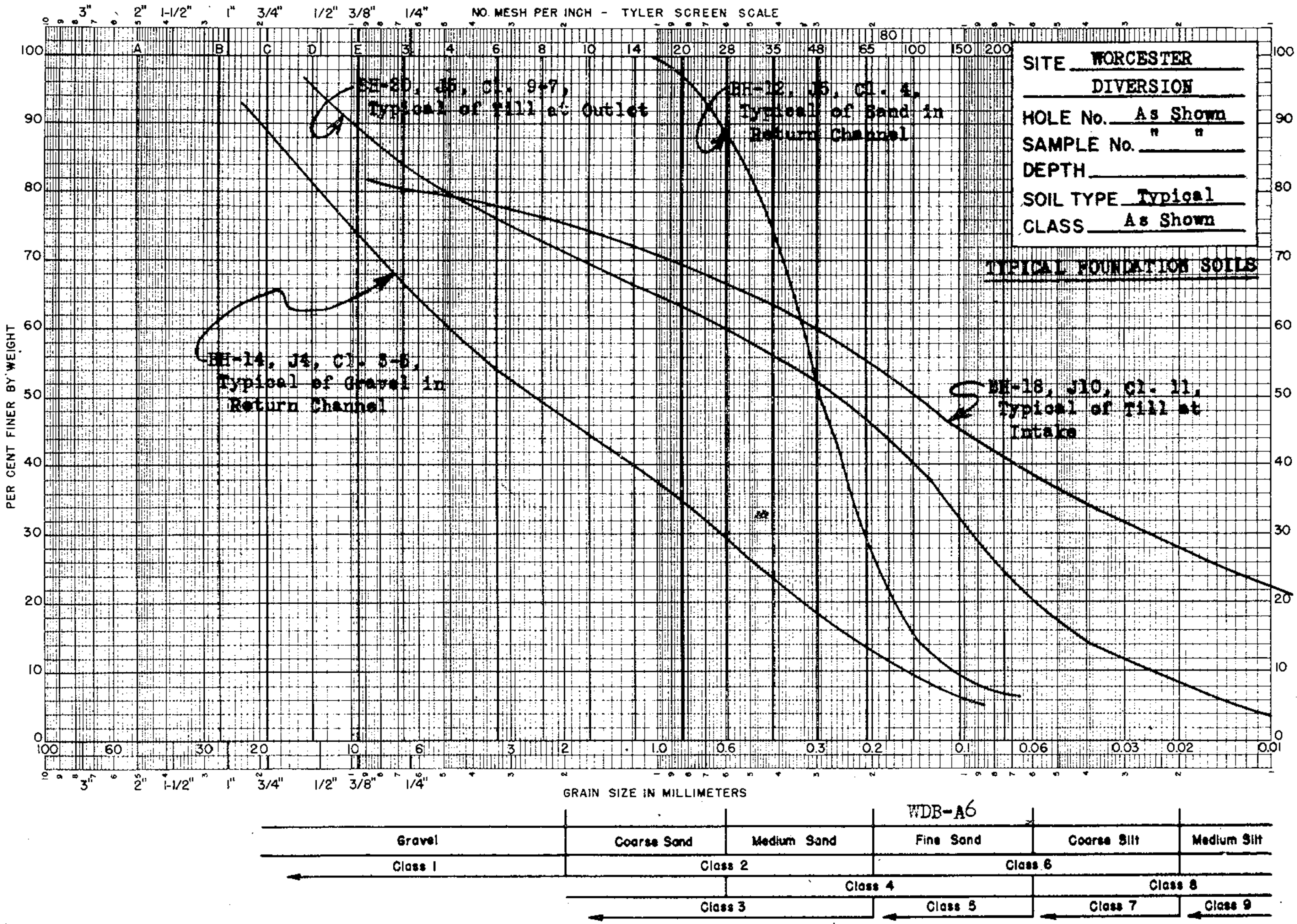


DIAGRAM SHOWING LIMITS OF SOIL CLASSES

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



APPENDIX III PLATE NO. 2

SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R. I.

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX IV

HYDRAULIC DESIGN

To accompany Definite Project Report

dated Sept. 1946

WORCESTER DIVERSION

APPENDIX IV - HYDRAULIC DESIGN

CONTENTS

WORCESTER DIVERSION

APPENDIX IV. HYDRAULIC DESIGN

PLATES

TABLES

APPENDIX IV. HYDRAULIC DESIGN

4-01. Hydraulics of Diversion Tunnel and Channel. - a. Criteria for Design. - (1) General. - The intake structure, gates, tunnel and channel which will constitute the Worcester Diversion were designed to pass the peak rate of discharge of the design flood, 5600 c.f.s., with the water surface in Leesville Pond at the spillway crest, elevation 484.5m.s.l.

(2) Design Flood. - Computation of the design flood for the Worcester Diversion is explained in detail in paragraph 1-03. It has a peak rate of discharge of 5600 c.f.s. and an average annual chance of occurrence of 0.2% according to the frequency-magnitude curve shown on Plate I-4.

(3) Tailwater Elevations in Blackstone River. - Development of a tailwater rating curve and the probable rate of discharge in the Blackstone River at the outlet end of the Worcester Diversion Channel are described in paragraph 1-03 c. To design the diversion channel for all rates of discharge under which it may operate, it was necessary to know the corresponding rates of discharge and tailwater elevations in the Blackstone River at the outlet end of the diversion channel. Upon the basis of the design storm conditions discussed in the paragraph referred to above, it was shown by computation that whenever the Worcester Diversion div erts the entire inflow to Leesville Pond the maximum rate of discharge in the Blackstone River at the lower end of the diversion channel will be twice the rate of discharge through the Diversion. If the design storm is concentrated almost entirely over the drainage area above the Diversion intake, the minimum probable rate of flow in the Blackstone River will result. This minimum rate of discharge was obtained by adding an assumed base flow of 100 c.f.s. to the diversion channel discharge. By the use of these assumptions and the tailwater rating curve for the Blackstone River, the maximum and minimum tailwater elevations in the river for any selected rate of discharge through the Worcester Diversion were obtained.

(4) Method of Operation. - The Worcester Diversion inlet gates will remain closed until it becomes obvious that discharge over the Leesville Pond spillway will contribute to damaging stages through the City of Worcester. The gates will then be opened sufficiently to lower the pool in Leesville Pond to the spillway crest and divert the entire Leesville Pond inflow through the tunnel. During this period

of full diversion, the gates will be operated to maintain the pool in Leesville Pond at or slightly below the Spillway crest. As soon as the flood has subsided sufficiently to permit discharge over the Leesville spillway without causing damages in the City of Worcester, the gates will be closed. The minimum rate of discharge at which the Worcester Diversion will operate will be governed by the channel capacity of the Blackstone River through Worcester from the Leesville Pond Dam to the mouth of Hull Brook and it will be about 500 c.f.s. as demonstrated by the following analysis of the channel capacity for this section of the river. Three damage reaches are involved - I, from Leesville Pond Dam to Curtis Brook Dam; II, from Curtis Pond Dam to Middle River Bridge, and III, from Middle River Bridge to the mouth of Hull Brook. The respective index stations for these damage reaches are (1) U.S.G.S. gaging station at Webster Street, Worcester, (2) the M. J. Whittall Association Dam and (3) The Millbury Woolen Mills Dam. Pertinent data regarding drainage areas and channel capacities at the index station and corresponding proportionate discharge rates past the Leesville Pond Dam are as follows:

Reach	D. A. At Index Station	Channel Capacity at Index Station	Corresponding Disch. at Leesville Dam (By ratio of D.A.'s)
I	31.3	700	700
II	49.9	1150	720
III	65.5	1190	570

From the foregoing table it is apparent that discharge through the Worcester Diversion will have to start when flow over the Leesville Pond Dam reaches 500 c.f.s. to avoid having discharge over this dam contribute to damage stages in the lower of the three damage reaches listed above.

Operation of the Diversion as described above will effect flood stages through Worcester as shown in the following table, for two typical floods, the "design flood" and the March 1936 flood. The "design flood" referred to in this table is the flood that would result from extension of the "design storm" described in paragraph 1-03 b over the whole drainage area tributary to the Blackstone River above the mouth of Hull Brook.

TABLE 1

REDUCTION OF FLOOD STAGES THROUGH WORCESTER

Location	Design Flood	March 1936 Flood
U.S.G.S. Gaging Station	12.9	8.0
Webster St. Lower Bridge	6.0	5.0
Below Dam, Hopeville Mfg. Co.	3.2	2.5
M. J. Whittall Association Dam	5.0	2.7
Southbridge St., Bridge	4.6	3.4
Below Central Works, Am. Steel & Wire Co.	3.4	2.7

When the Diversion is in operation the river stage below the mouth of Hull Brook may be slightly higher than would naturally occur if the Diversion was not in operation, due to flow through the Diversion being more rapid than by the natural channel. Because of the relatively short length of natural channel involved and the small amount of valley storage within this section of river channel any increase in stage would be negligible and for many storms there would be no increase in stage.

b. Details of Hydraulic Design. - (1) Intake. - (a) Approach. - Under normal operating conditions, as stated in the preceding paragraph, a simple riprapped approach channel would be adequate. If, however, the gates should remain fully open after the peak inflow to Leesville Pond had passed, the pond would be drawn down rapidly, and erosion and displacement of riprap might result. Such an occurrence probably would necessitate expensive clean-up work in the tunnel as well as repairs to the approach channel. To avoid this possibility a control weir 101 feet long with crest at elevation 478.0 m.s.l. is provided just upstream from the tunnel inlet. If the gates are fully open there will be free discharge over the weir for flows up to about 5000 c.f.s. At the design discharge the weir will be approximately 55% submerged. Free discharge over the weir would, however, be avoided by proper gate operation.

(b) Gates. - For any given number and size of gates, the sill elevation is determined by the requirement that 3920 c.f.s. must be passed with one gate closed and the water in Leesville Pond at elevation 484.5. The most economical design was found to be three gates, 6 feet wide by 12 feet high. Because of foundation conditions the sills were set at elevation 455.0, which is somewhat lower than necessary to meet the capacity requirement. The computed capacity with one gate closed and the water in Leesville Pond at elevation 484.5 is 4300 c.f.s.

(2) Tunnel. - The tunnel was designed to flow full at the design discharge of 5600 c.f.s. with the water surface in Leesville Pond at elevation 484.5. The invert at the exit portal was set at elevation 407.0, which is the lowest elevation consistent with adequate slope in the open channel. Friction loss in the tunnel, which will be about 4500 feet long, was computed by the Manning formula with n = .013. Under these conditions a circular tunnel having a diameter of 16.0 feet was required. When the discharge is less than 5600 c.f.s. the tunnel will flow partly full.

(3) Stilling Basin. - A stilling basin just downstream from the tunnel portal was found to be necessary to insure non-erosive velocities in the open channel after it runs out of rock cut about 600 feet downstream from the tunnel portal. The design of the stilling basin was governed by the tailwater depth necessary to produce the hydraulic jump with maximum tunnel discharge and corresponding minimum water surface elevation in the Blackstone River. Two methods of obtaining the required tailwater depth were feasible, (1) excavation of the channel bottom and (2) construction of a sill across the channel. In order to obtain the required depth entirely by excavation, it would have been necessary to set the floor of the stilling basin about 5 feet below the bottom of the channel. This scheme would have created a permanent pool

which could not be drained, and was rejected for that reason. Because of the relatively low kineticity of the flow entering the basin, a jump of poor quality would have resulted from use of a sill alone to provide the required depth. A basin in which a satisfactory jump will form and which can be drained was designed by depressing the floor 2 feet below the channel bottom and providing a sill to ensure the necessary depth of tailwater. The basin will be drained by means of a pipe through the end sill. A drainage channel will be excavated in the rock to connect the pipe with the low part of the open channel section excavated in earth. The quality of the jump in this basin will not be so good as that in a basin designed by the first method but will be better than that in a basin using an end sill alone. Details of the stilling basin are shown on Plate No. IV-1.

(4) Open Channel. - Water-surface profiles in the open channel were computed for the purpose of determining, over the full range of diversion discharges from 500 c.f.s. to 5600 c.f.s., the following maxima and minima:

(a) Stilling basin tailwater elevations.

(b) Velocities and water surface elevations in the open channel.

Assumed conditions for these computations were:

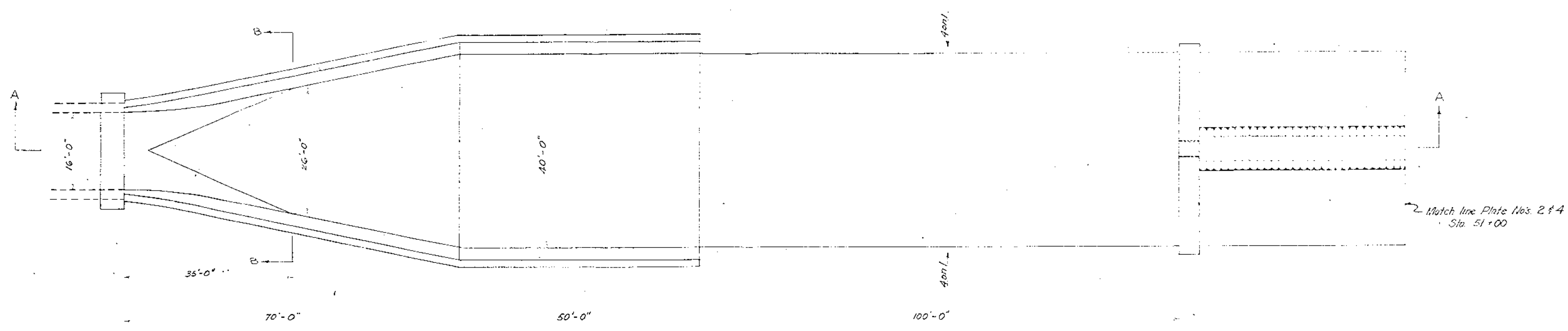
(a) Maximum river elevation and maximum friction coefficient.

(b) Minimum river elevation and minimum friction coefficient. The resulting computed profiles and velocities are shown on Plates Nos. IV-2 to IV-5, inclusive. Stilling basin tailwater curves are shown on Plate No. IV-1.

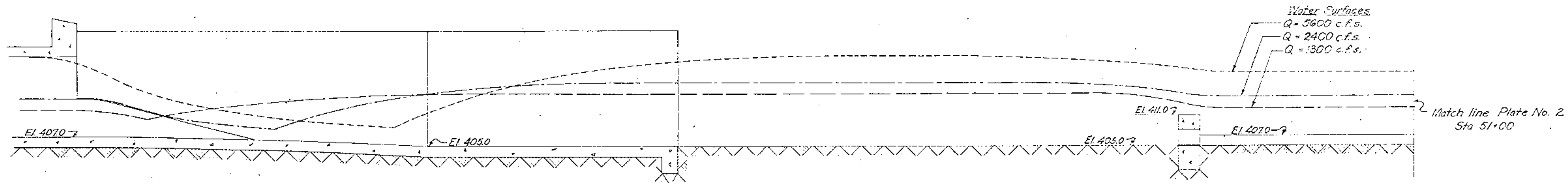
4-02. Leesville Spillway Hydraulics. - The maximum safe capacity of the Leesville Dam spillway is estimated to be about 1500 c.f.s. To pass this discharge over the 82-foot spillway, the pool would have to be up to the top of the abutments which are only 3.3 feet above the spillway crest. With the pool at this elevation, the Worcester Diversion would discharge about 5900 c.f.s., and the rate of inflow to Leesville Pond would be about 7400 c.f.s. Storage between the spillway crest and the top of the dam amounts to about 480 acre feet.

Information from the owners of Leesville Dam indicates that there is probably no core wall in the dam embankments at the ends of the spillway and that the embankment material is random earth and gravel fill which would wash out very quickly if the dam were overtopped. This was demonstrated in March 1936 when a peak discharge of 2520 c.f.s. washed out part of the embankment back of the right abutment.

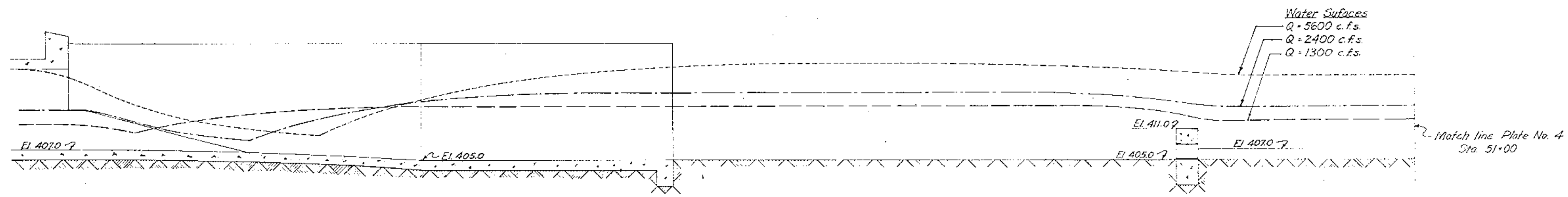
A computed spillway flood having a peak discharge of 20,900 c.f.s. as computed in paragraph 1-04 d would overflow the banks of Leesville Pond. A large part of such a flood would reach the City of Worcester by way of a railroad cut and highway. Leesville Dam probably would be overtopped. However, even if the dam were rendered ineffective as a control, which is unlikely in view of the nature of its construction, existing controls downstream from the dam would cause the water surface in Leesville Pond to remain sufficiently high for the Worcester Diversion to operate at design capacity or more until the peak of the flood had passed. Therefore, it is considered unnecessary to modify in any way the existing Leesville Dam and no surcharge-length relations were developed for the computed spillway flood.



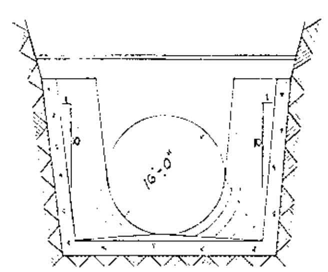
PLAN



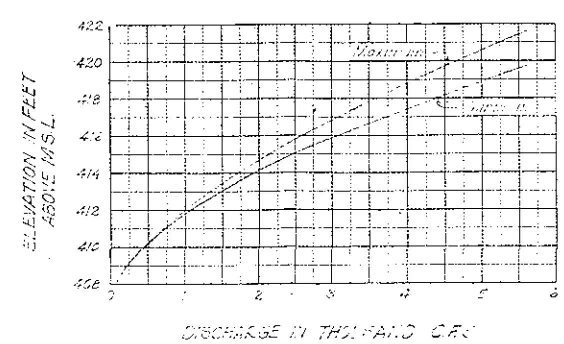
SECTION A-A MINIMUM TAILWATER



SECTION A-A MAXIMUM TAILWATER



SECTION B-B



TAILWATER RATING CURVES

STA. 51+00

BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

STILLING BASIN

HYDRAULIC DESIGN

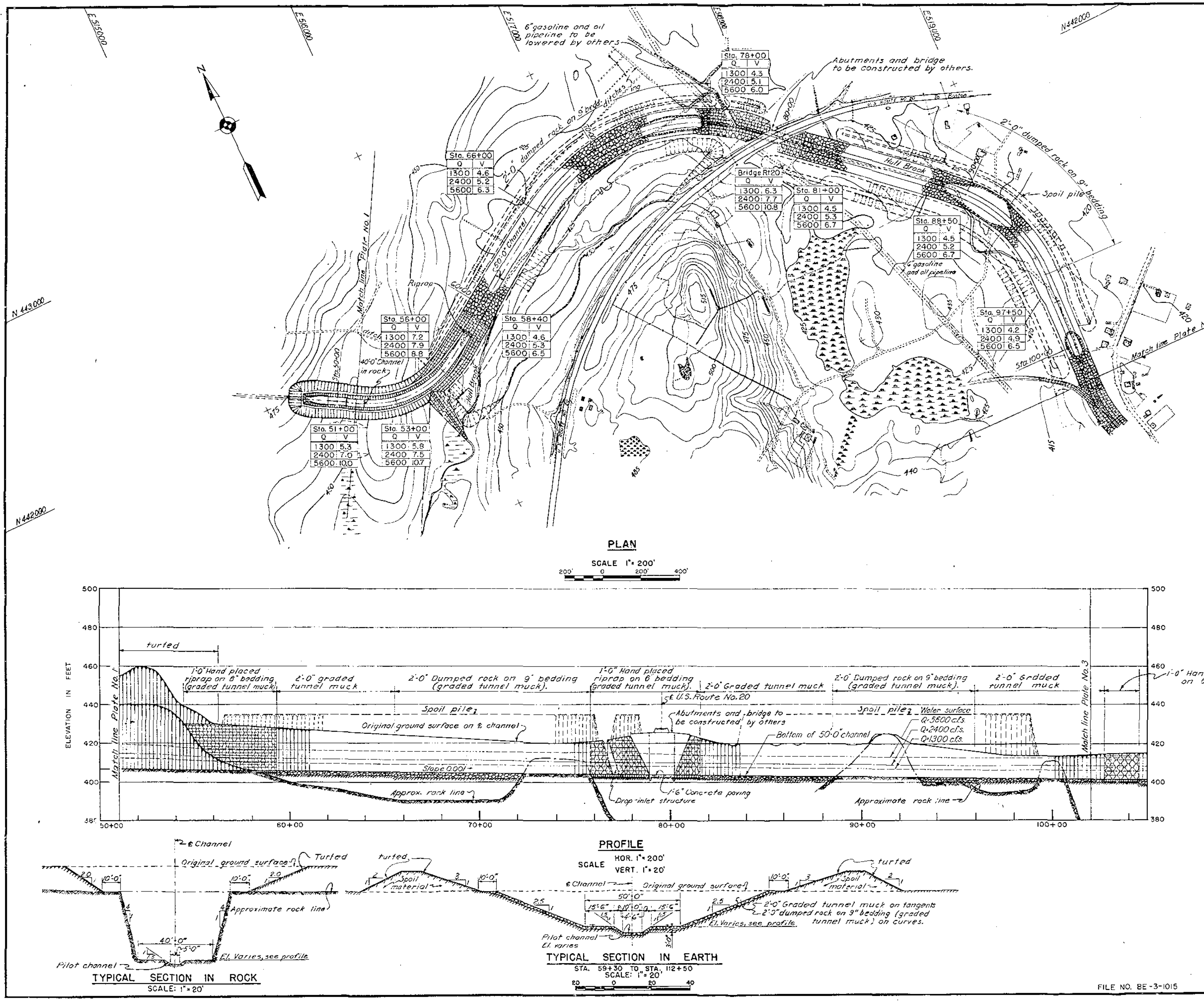
BLACKSTONE RIVER MASSACHUSETTS

U.S. ENGINEER OFFICE PROVIDENCE, R. I.

SCALE: 1" = 10'

10 0 10 20

JULY, 1946



BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

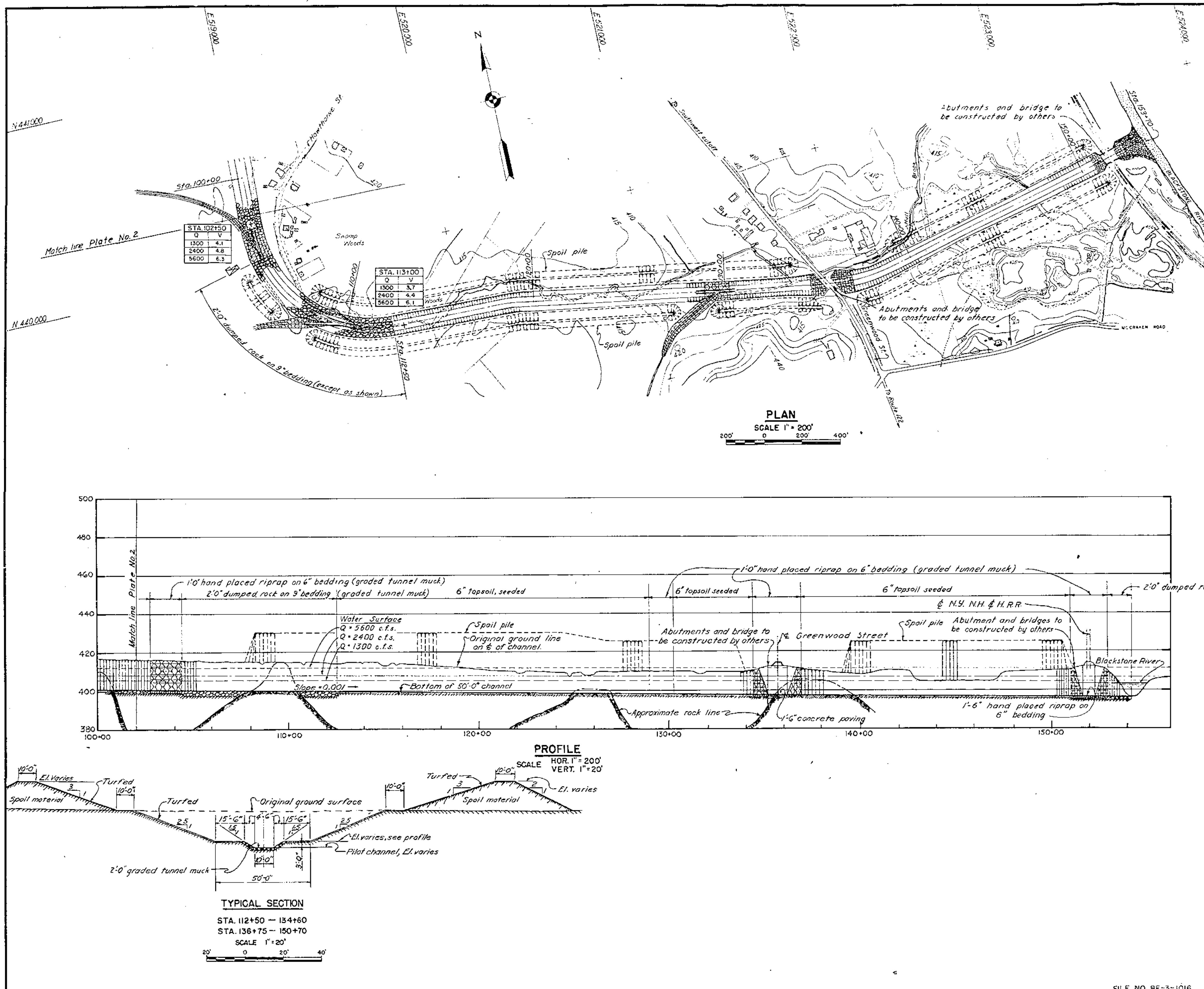
WATER SURFACE PROFILES AND COMPUTED VELOCITIES

MINIMUM TAILWATER

BLACKSTONE RIVER — MASSACHUSETTS

U.S. ENGINEER OFFICE — PROVIDENCE, R.I.

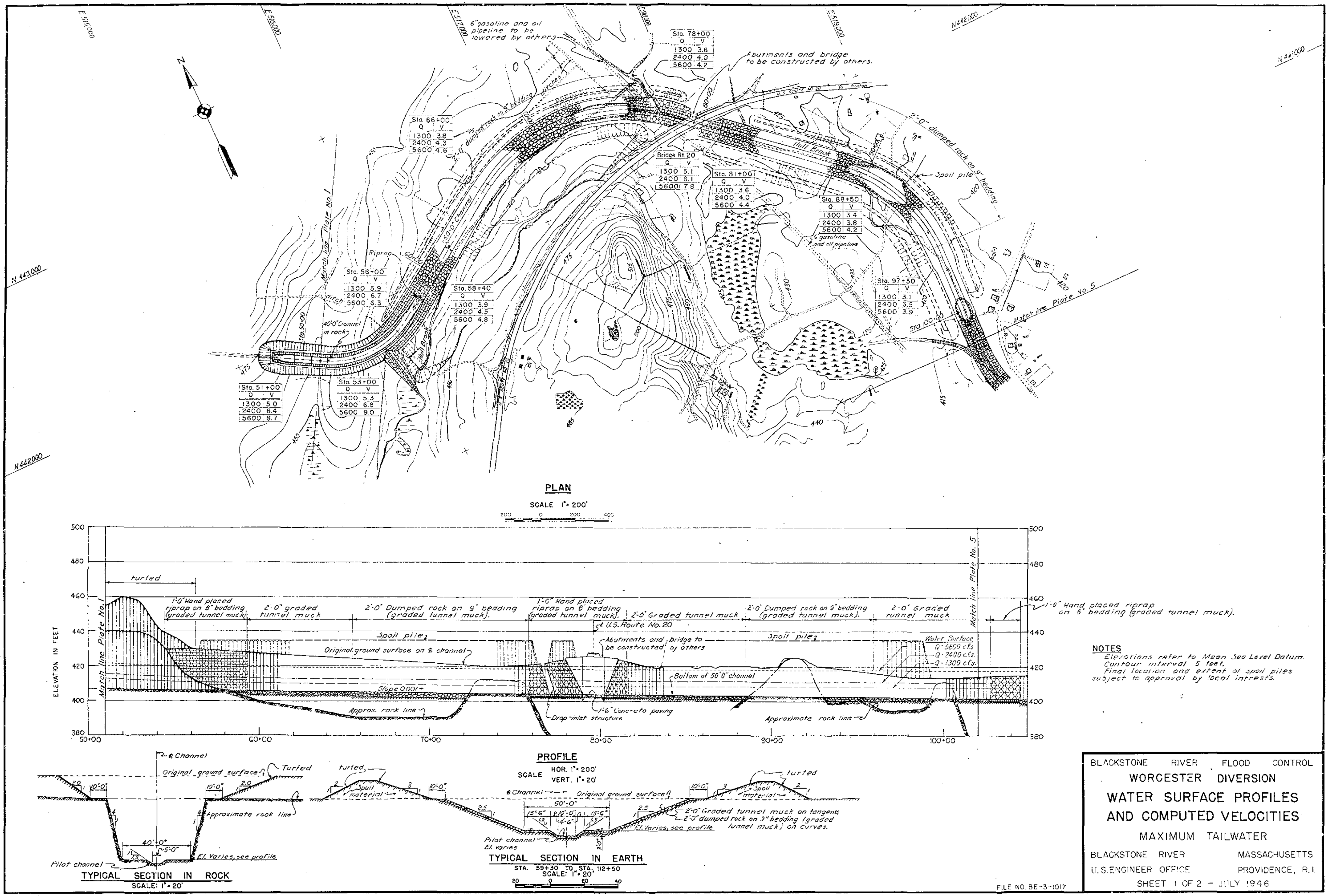
SHEET 1 OF 2 – JULY, 1946



NOTES
Elevations refer to Mean Sea Level Datum.
Contour interval 5 feet.

FILE NO. 8E-3-1016

BLACKSTONE RIVER FLOOD CONTROL
WORCESTER DIVERSION
WATER SURFACE PROFILES AND COMPUTED VELOCITIES
MINIMUM TAILWATER
BLACKSTONE RIVER MASSACHUSETTS
U.S. ENGINEER OFFICE PROVIDENCE, R.I.
SHEET 2 OF 2 – JULY 1946



BLACKSTONE RIVER FLOOD CONTROL

WORCESTER DIVERSION

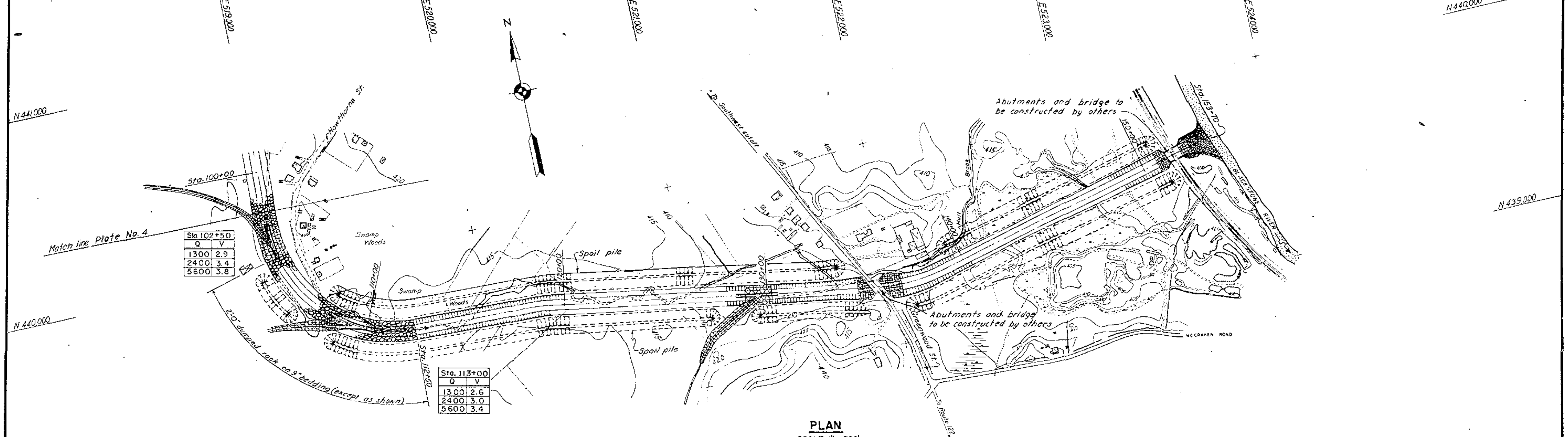
WATER SURFACE PROFILES AND COMPUTED VELOCITIES

MAXIMUM TAILWATER

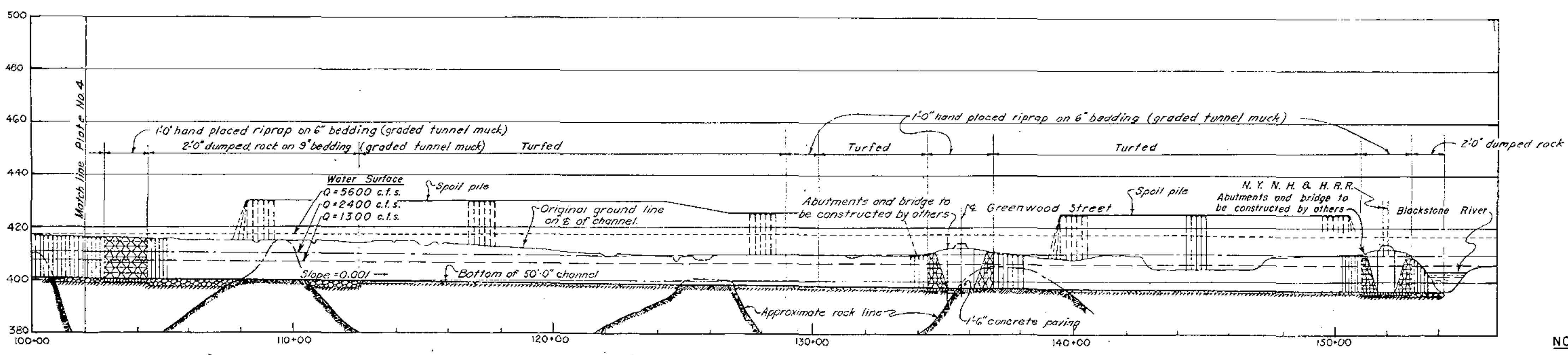
BLACKSTONE RIVER MASSACHUSETTS

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

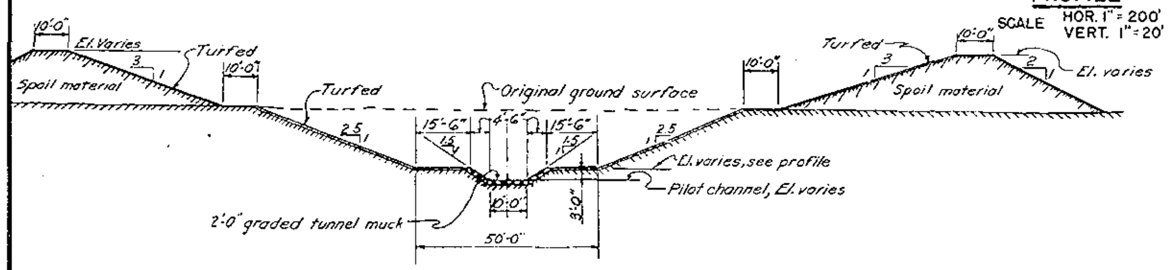
SHEET 1 OF 2 – JULY 1946



PLAN
SCALE 1" = 200'



PROFILE
SCALE HOR. 1" = 200' VERT. 1" = 20'



TYPICAL SECTION
STA. 112+50 — 134+60
STA. 136+75 — 150+70
SCALE 1" = 20'

NOTES
Elevations refer to Mean Sea Level Datum.
Contour interval 5 feet.

BLACKSTONE RIVER FLOOD CONTROL
WORCESTER DIVERSION
WATER SURFACE PROFILES AND COMPUTED VELOCITIES
MAXIMUM TAILWATER
BLACKSTONE RIVER MASSACHUSETTS
U. S. ENGINEER OFFICE PROVIDENCE, R. I.
SHEET 2 OF 2 — JULY 1946

FILE NO. BE-3-1018

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX V

STRUCTURAL DESIGN

To accompany Definite Project Report

dated Sept. 1946

WORCESTER DIVERSION

APPENDIX V - STRUCTURAL DESIGN

CONTENTS

APPENDIX V. STRUCTURAL DESIGN

5-01. Design Standards. - Standard practice in structural design has been followed throughout. In general, the concrete design has been based on the "Joint Code of the American Concrete Institute and the Reinforcing Steel Institute for the Design of Concrete and Reinforced Concrete" issued in 1941 and the Joint Committee; Recommended Practice for Concrete and Reinforced Concrete, issued in 1940. Allowable stresses are reduced to insure added permanence and safety in accordance with the character of the project.

5-02. Loadings. - The following unit weights have been assumed and used throughout for structural design:

Material	Unit Weight Pounds per cu. ft.
Water	62.5
Concrete, plain or reinforced	150
Steel	490
Dry Earth	100
Saturated Earth	125
Rock	175

Horizontal earth pressures were taken as 35 lbs. per square foot equivalent fluid pressure for dry earth and 80 lbs. per square foot for saturated earth.

5-03. Tunnel Sections. - Concrete tunnel sections are proposed for economy of construction consistent with good engineering practice. The non-reinforced tunnel section in rock is designed for erosion and wear as well as structural strength. In view of the expected intermittent use, the 28 ft. per second and the highly abrasive properties of sands encountered in this vicinity have been discounted in design. The reinforced tunnel section in earth is designed for the total earth load above the structure and includes an allowance for erosion.

5-04. Structures. - Intake Structure. - The control house substructure is designed to withstand full surcharge hydrostatic load on the upstream face and foundation (El. 484.5 M.S.L.) and to withstand the dead loads of the structure and surrounding earth. The structure is founded on till of suitable quality. The control house floor and floor beams are designed for full dead load and a live load of 500 pounds per square foot, assuming full continuity at the ends. The stairs are designed for a live load of 125 pounds per square foot of horizontal surface plus full dead load using a span equal to the horizontal distance, and designing the positive steel for simple beam moment and the negative steel for fixed beam moment.

5-05. Weir and Wing Walls. - The weir and floor slab are designed for full hydrostatic uplift for maximum flood throughout as well as earth pressures. The retaining walls are designed by use of the Coulomb theory in preference to the Rankine theory because of the surcharge behind the walls and an inclined concrete surface in contact with the fill. This procedure is believed to give results more in conformity with observed data.

5-06. Slope Treatment of Channel. - Several types of slope protection treatments have been selected for use on the following designated sections of the channel and are shown on Plates Nos. 5 and 6 of the report.

a. Intake Channel. - The slopes will be protected against wave action and erosion from approach channel velocities of 5-1/2 ft. per sec. by 12 inches of hand-placed riprap on 6 inches of suitable bedding.

b. Return Channel will be protected against erosion from channel velocities of 6 ft. per sec. in channel sections and somewhat higher velocities at bridges and against frost sloughing by the following treatments:

(1) 12 inches of hand-placed riprap on 6 inches of suitable bedding under and adjacent to all bridges except where a concrete bottom is provided.

(2) 24 inches of dumped rock riprap at the outfall into the Blackstone River and extending to the hand-placed riprap at the N.Y., N.H. & H. RR bridge--placed largely under water and, therefore, without bedding.

(3) 24 inches of tunnel muck filter blanket on the tangents in the lake bed area. In addition to resisting erosion, this blanket is intended to furnish protection against frost sloughing of slopes during spring thaws by decreasing the depth of frost penetration into the silts and also lengthening the period of thawing in the spring, thus permitting the excess water to escape through the more pervious blanket.

(4) The major curves will have slopes protected by 24" of dumped rock from channel rock excavation on 9" of bedding. This will furnish protection both against channel currents and frost sloughing during spring thaws.

(5) Channel slopes, except where protected with dumped rock, riprap or tunnel muck, and surfaces of spoil piles will be mulched and seeded. Where exposed material is unsuitable to support growth a layer of topsoil not less than 6 inches on channel slopes and 3 inches on spoil piles will be placed before mulching and seeding.

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX VI

HYDROELECTRIC POWER

To Accompany Definite Project Report
dated Sept. 1946

WORCESTER DIVERSION

APPENDIX VI. HYDROELECTRIC POWER

The development of hydroelectric power is not practical on a project of this type.

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX VII-A

RELOCATIONS

To accompany Definite Project Report

dated Sept. 1946

WORCESTER DIVERSION

APPENDIX VII-A - RELOCATIONS

CONTENTS

APPENDIX VII-A - RELOCATIONS

7A-01. General. - Investigations relative to necessary relocations of highways, public utilities and other improvements were made on the basis that the City of Worcester would provide facilities equal to facilities now existing wherever the channel or tunnel interfered with existing rights of way.

7A-02. Highways. - Four highways cross the project alignment. In order proceeding downstream from the intake, the following facilities will be required:

a. State Route 12. - No bridge or other facility will be required. The diversion at this location will be in tunnel with invert grade 51 feet below the road. No disturbance to the road during construction of the tunnel will be necessary.

b. U. S. Route 20. - The diversion will pass under U. S. Route 20 in an open channel following the present alignment of Hull Brook. A new bridge will be required to replace the existing Hull Brook culvert. The bridge will require a clear span of 50 feet and a clearance from channel bottom at elevation 404.0 to bridge floor at El. 426± of 22 feet. No changes to the highway grade will be required. The maximum water surface elevation in the channel at this location will be 418.6. The estimate of cost for this structure is based on a 50-foot span, reinforced concrete, deck girder for the full width of the roadway designed for H20 loading.

c. Hawthorne Street. - The diversion channel will cross Hawthrone Street, a Millbury town road. No present need exists for providing a bridge at this location in view of the fact that the single dwelling thereby cut off from access will be among the properties acquired for the channel right-of-way.

d. Greenwood Street. - The diversion channel will pass under Greenwood Street following the present alignment of Hull Brook. A new bridge will be required to replace the existing Hull Brook bridge. The bridge will require a clear span of 50 feet and a minimum clearance from channel bottom at el. 398.5 to bridge floor at el. 416.0± of 17.5 feet. Minor changes to highway grades will be necessary. The maximum water surface elevation in the channel at this location will be 417.3. The estimate of cost for this structure is based on a 50-foot span, steel beam with reinforced concrete deck for the full width of the roadway designed for an H-12 loading.

7A-03. Proposed Highway Relocations. - No highway relocations are proposed for this project.

a. Summary of Proposed Highway Alterations. - Recommended highway alterations consist only of the two new bridges and minor modifications in approach pavements as described in the preceding paragraphs and shown on Plate 3 of the report.

7A-04. Railroads. - Two branch lines of the New York, New Haven and Hartford Railroad are crossed at the Worcester Diversion. The Worcester-New London Branch, parallel to Leesville Pond, will not require relocation as the result of this project. Construction at this railroad right-of-way will consist of tunneling in earth and existing railroad facilities will not be disturbed. The Providence-Worcester Branch of the New Haven Railroad is crossed near the outlet and a new bridge will be installed at the expense of the City of Worcester. The estimate of cost for this structure is based on a 50-foot span, double track, open deck, plate girder bridge designed for an E-60 loading.

7A-05. Utilities. - The Worcester Diversion Project will have no effect on existing utilities except an 8-inch water pipe line in Greenwood Street which is to be bridged and the 6-inch petroleum pipe line near U.S. Route 20. Adequate provision will be made to carry pipe lines on structures or depress them under the channel. For the purposes of this report, both pipe lines are assumed lowered to 4 feet below finish grade on their present alignment.

7A-06. Cemeteries. - No cemetery relocation will be required as the result of the construction of the Worcester Diversion.

7A-07. Cost Summaries. - Cost estimates exclusive of land damages have been made from reconnaissance and considered in discussion with local authorities, on the basis of the relocations described in the preceding paragraphs and are tabulated below:

TABLE

Route No. 20 Highway Bridge	$75,000
Greenwood St. Bridge	25,000
Railroad Bridge	75,000
Care of Pipe Lines	2,500
	$177,500
Contingencies + 20%	35,500
	213,000
Engineering + 15%	32,000
Estimated Total Cost for Bridges and Utilities	$245,000

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX VII-B

REAL ESTATE PLANNING

To accompany Definite Project Report

dated Sept. 1946

APPENDIX VII-B. REAL ESTATE PLANNING

CONTENTS

TABLES

APPENDIX VII-B. REAL ESTATE PLANNING

7B-01. General. - The estimated cost of the value of real estate situated in the channel and intake area was determined on the basis of acequisition in fee simple by the City of Worcester for those items which must be maintained permanently; in the tunnel area acquisition will be by easement and for spoil area right of entry will be by easement or permit. In estimating the cost of the real estate involved due consideration was given to acquiring lands to avoid the payment of excessive severance damages. The estimate of the value of the property to be acquired was determined by inspection of the area by representatives of this department since no such estimate has as yet been prepared by local officials.

7B-02. Description of the General Area. - The Worcester Diversion extends from Leesville Pond in the Town of Auburn to the Blackstone River at the mouth of Hull Brook in the Town of Millbury, both in Worcester County, Massachusetts. The Towns of Auburn and Millbury are situated south of the City of Worcester. Within the area traversed by the diversion the development is characteristic of rural and suburban communities. The portion traversed by the Diversion is generally agricultural and the acquisition will be through pasture land. Massachusetts State Highway Route #12, Mass. U. S. Route #20 and Greenwood Street in Millbury traverse the area through which the diversion tunnel and channel is to be constructed. The New York, New Haven and Hartford Railroad traverses the area at two places. The topography overlying the tunnel is hilly and sparsely populated. The diversion channel passes through the towns of Auburn and Millbury approximately following the course of Hull Brook. The land is partially wooded with gently rolling topography. Some farm units and residential buildings are located near the channel.

7B-03. Classification of Real Estate Affected. - a. The classification of land to be taken in fee is as follows:

Cottage lots	4 Acres
Cultivated	19 "
Pasture	9 "
Woods	3 "
Low wet pasture	5 "
Gravel pit	2 "

b. The classification of land on which permits or easements are to be taken is as follows:

Cottage lots	6 Acres
Cultivated	17 "
Pasture	19 "
Woods	8 "
Low wet pasture	4 "
Gravel pit	4 "

7B-04. Water Main. - There is an 8 inch water main under the present bridge on Greenwood Street which will be relocated to a similar position on a new bridge to be constructed by others. There will be no land requirements for this relocation.

7B-05. Petroleum Pipe Line. - A 6-inch petroleum pipe line owned by the Socony-Vacuum Oil Company traverses the area under an existing bridge on Route #20. The pipe line will be lowered in grade on its present right of way and an easement or permit will be required to cross this right of way.

7B-06. Additional Easements. - A right of way will be required over an existing road leading from Route #12 to the work area at the entrance portal of the tunnel with the right to improve said road. (Easements or permits will be obtained for the construction of the tunnel.)

7B-07. Water Rights. - The Consolidated Rendering Company of Boston, Massachusetts, owns the dam and water rights around Leesville Pond. There is no present power development. The water from the pond is used in processing at the local rendering plant. No diversion of water from this pond will be performed except during floods. The diversion of water occurs only during flood periods and during such periods will be controlled by the gates.

7B-08. Mineral Rights. - There is no evidence that there are any existing sub-surface rights for coal, oil, gas or minerals in the area to be affected. There is however the probability that the well water supply of properties on the surface of Packachoag Hill above the tunnel may be temporarily affected. If this condition arises, the owners would be required to file claims against the City of Worcester in order to compensate for any loss. No allowance is made for such claims in this report.

7B-09. Cemeteries. - There are no cemeteries located in the area to be acquired.

7B-10. Land Acquisition Cost Estimate. - The estimate of cost for lands, improvements and acquisition costs are tabulated below.

TABLE I - REAL ESTATE PLANNING

ESTIMATE OF COSTS FOR LANDS, IMPROVEMENTS

AND ACQUISITION EXPENSES

Item	Cost
Land and Improvements (Fee simple and Easements)	$60,000
Contingencies plus 10%	6,000
	$66,000
Acquisition Expenses plus 15%	10,000
Total Acquisition Costs	$76,000

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX VIII

AVAILABILITY OF CONSTRUCTION MATERIALS

To accompany Definite Project Report

dated Sept. 1946

WORCESTER DIVERSION

APPENDIX VIII. AVAILABILITY OF CONSTRUCTION MATERIALS

CONTENTS

PLATES

WORCESTER DIVERSION

APPENDIX VIII. AVAILABILITY OF CONSTRUCTION MATERIAL

8-01. General. - Ample material for lining the channel bottom and slopes as indicated on Plates Nos. 4, 5 and 6 of the report are available from the proposed tunnel and channel excavation. The earth excavation is to be spoiled adjacent to the channel as indicated on Plates Nos. 5 and 6.

8-02. Rock. - a. Rock for riprap will be obtained from rock excavation in the channel which is somewhat harder than other rock in the project (See Appendix II). It is estimated that approximately 80,000 cu. yds. will be available to meet the requirement of 8,000 cu. yds. of hand-place riprap and 18,000 cu. yds. of dumped rock.

b. Material for the 24-inch tunnel muck filter blanket will be obtained by washing excess fines from the tunnel muck (See Plate No. VIII-1). It is estimated that it will be necessary to remove by washing 50 percent of the material finer than #200-mesh sieve to provide a filter blanket which will be more pervious than the lake bed soils. It is estimated that approximately 40,500 cu. yds. of tunnel muck will be available for this processing and should yield the requirement of 30,800 cu. yds. of filter blanket material. Assumptions as to gradation of tunnel muck are based on experience with similar materials in the Quabbin tunnel.

(1) It is also proposed to allow use of processed tunnel muck as bedding for riprap as an alternate to bank-run gravel, which latter is not abundant nearby. For this purpose it will be necessary to remove excess fines from the tunnel muck and also to remove rock fragments larger than 3 inches. Approximately 10,800 cu. yds. of bedding will be required.

8-03. Topsoil. - Topsoil for development of turf on slopes of channel or spoil banks not protected by riprap or filter blanket and at locations where natural soil will not support growth will be obtained from stripping of channel area. Ample material is available but slight selection must be exercised to avoid inclusion of highly acidic swamp muck or other objectionable materials.

8-04. Concrete Aggregates. - Processed sand and gravel are obtainable at a commercial gravel and transit-mix concrete plant about 1/2 mile from intake end of the tunnel, which have a satisfactory service record and have appeared suitable in preliminary tests. Other gravel deposits are located at about a mile or more haul but do not have plant facilities.

8-05. Summary of Materials Available. - In balancing materials required against those available, Table No. VIII-1 forms a preliminary usage plan, which is designed to give a conservative estimate of wastage and losses.

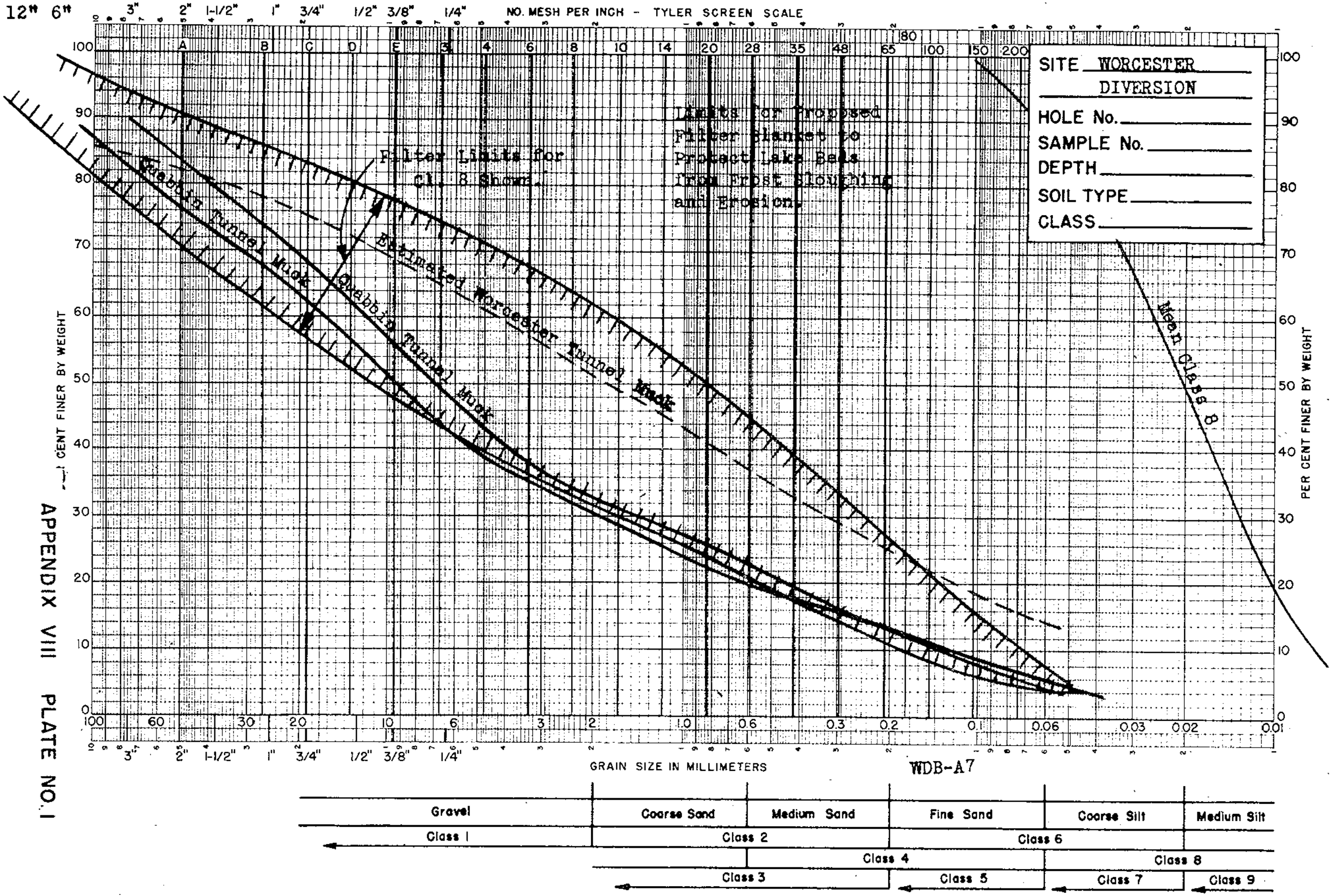
Table No. 1

Summary of Materials Required and Available, Worcester Diversion

Type	Embankment measure cu.yds.	Shrinkage & Loss Factor	Excavation measure cu.yds.	Source	Estimated Total Available cu.yds.	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Riprap	8,000	1.2	6,700	Channel	80,000	Remainder to be Wasted
Dumped Rock	18,000	1.3	13,900	do		
Graded Tunnel Muck	20,000	1.0	20,000	Tunnel Muck	40,500	
Bedding	10,800	1.0	10,800	do		
Topsoil	18,00	0.85	21,200	Stripping	50,000	

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

APPENDIX VIII PLATE NO. I

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
PROVIDENCE, RHODE ISLAND

BLACKSTONE RIVER FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT

WORCESTER DIVERSION

APPENDIX IX

SPECIAL STUDIES

To accompany Definite Project Report

dated **Sept.** 1946

WORCESTER DIVERSION

APPENDIX IX - SPECIAL STUDIES

CONTENTS

APPENDIX IX - SPECIAL STUDIES

9-01. Selection of Definite Project Plan. - In arriving at a selection of the most economical and locally acceptable overall plan of development for the site within the scope of the authorization and subsequent instructions, several factors were considered of salient importance.

(1) Providing the authorized flood control capacity.

(2) Limiting interference to existing development near the affected area to a minimum.

(3) Avoiding claims and excessive damages from lowered ground water tables wherever possible.

9-02. Alternate Studies. - Having eliminated the possibility of alternate methods of overall development, (See Sec. 9), the foundation of the site was investigated by means of borings and seismic explorations. A rock contour map was prepared and various alignments of tunnel and channel studied. The most economical outlet portal location was determined and is shown on Plate No. 4.

9-03. Board of Consultants. - a. The plan was presented to the Board of Consultants consisting of Messrs. W. H. McAlpine, J. D. Justin, W. F. Uhl, and F. E. Fahlquist. There follows a transcript of the minutes of the consultants meeting at which the project was discussed:

Worcester Diversion. - The party visited the Worcester Diversion project on 9 April 1946 and inspected the intake site and route of the Diversion.

The general design for the Worcester Diversion project was discussed in detail at the meeting held in the District Office on 10 April 1946. The basic physical conditions and the design as proposed by the District Office were outlined by representatives of the Planning Branch, Hydrology and Reports Branch, Structural Section and Soils Laboratory. A question-and-answer discussion was then held with respect to the major features of the design and investigations to date and additional planning necessary for completion of the Definite Project Report.

It was the opinion of the consultants that the explorations and data were sufficiently complete to recommend a definite project for this site and that the hydrologic and hydraulic criteria were sufficient to proceed. Messrs. Hathaway and Winslow considered that additional hydraulic studies should be made to improve the intake and to study the need for a stilling basin at the outlet portal. Messrs. McAlpine, Justin and Uhl expressed doubt as to the need for a stilling basin.

The foundations for the tunnel were considered by the Board and it was agreed that sufficient explorations had been made for the purposes of the definite project report. Mr. Fahlquist was of the opinion that the present location of the westerly (intake) section of the tunnel is the best that can be found for minimum length of tunnel and greatest economy and stated that the overburden through which the westerly 600 feet of tunnel will be constructed is a very compact glacial till (class 9 and 11). Mr. Fahlquist considered the rock line sufficiently well defined for present purposes and commented that, from inspection of the hillside along Route 12, there appeared to be no possibility of a more economical location where all tunnel construction would be in rock. Mr. McAlpine would prefer to have the intake structure and the tunnel in earth section constructed on rock or resting on pillars carried to rock and suggested that additional investigations be made covering that possibility. Allowance for steel sheet piling in estimating the unwatering of the intake was also suggested. Some concern was expressed regarding the stability of the slopes in the dumped riprap section of the channel. It was agreed that small failures, which could be corrected by annual maintenance, might occur but that the design as proposed would be satisfactory for the purposes of the definite project report.

The Board considered that the design of the tunnel was conservative. Messrs. Justin and Uhl considered that the lining of the tunnel in rock should not be more than 12" thick. It was agreed that the length of monoliths in the tunnel should not exceed 20 feet. A flexible joint between the tunnel sections in earth and rock was recommended. The Board agreed that grouting around the tunnel in rock should be required throughout.

The Board considered the problem of trash racks at the intake and was divided as to the necessity therefor. Mr. McAlpine favored a boom instead of a trash rack and Mr. Uhl suggested widely spaced rack bars. With regard to allowable velocities over riprap at the intake structure, Mr. McAlpine felt that such protection should be considered adequate for velocities not exceeding eight to ten feet per second. Mr. Uhl concurred. Mr. Justin thought that the maximum velocity for such protection should be limited to six feet per second, and recommended that the concrete paving on the bottom be extended to the end of the intake training walls.

With reference to a question raised regarding possible silting of the intake structure, the Board generally agreed that a minor amount of silting might take place but did not consider the problem sufficiently serious to justify any special preventive construction. It was felt that periodic sluicing through the gates would ordinarily remove the minor silt deposits encountered. Mr. McAlpine felt that elimination of the trash racks would tend to improve the situation as regards possible silting.

The possibility of drawdown of domestic wells in the vicinity of the tunnel was pointed out. The Board concurred in the desirability of the installation of groundwater observation wells for obtaining data as proposed by this office. Mr. Fahlquist considered that measures necessary to prevent drawdown of wells during tunnel construction would be impracticable and expensive.

b. Modification of design of intake and stilling basin was discussed with Mr. Uhl on 16 July 1946. The design was approved as presented.